Chapter 7

Substructure Design

To design a substructure properly, a designer must know or assume the loads acting on the substructure. Assumed loads must be verified before completing the final design. Since many loads can act on the substructure, not all loads will act with full intensity at the same time. To apply the appropriate loads to the substructure design, the designer must determine the critical combination of loads for given conditions.

ABUTMENT DESIGN PROCEDURE

7-1. There are separate design procedures depending on the type of load acting on the abutment. Determine the vertical or horizontal load as discussed below.

VERTICAL LOADS

7-2. For abutments constructed of piles or posts and footings, determine the total loads acting on the entire abutment. For continuous abutments (such as mass or reinforced concrete) *(Figure 7-1)*, determine the load per foot of abutment length. The dead load acts vertically through the centerline of the bearing plate *(Figure 7-2, page 7-2)*.

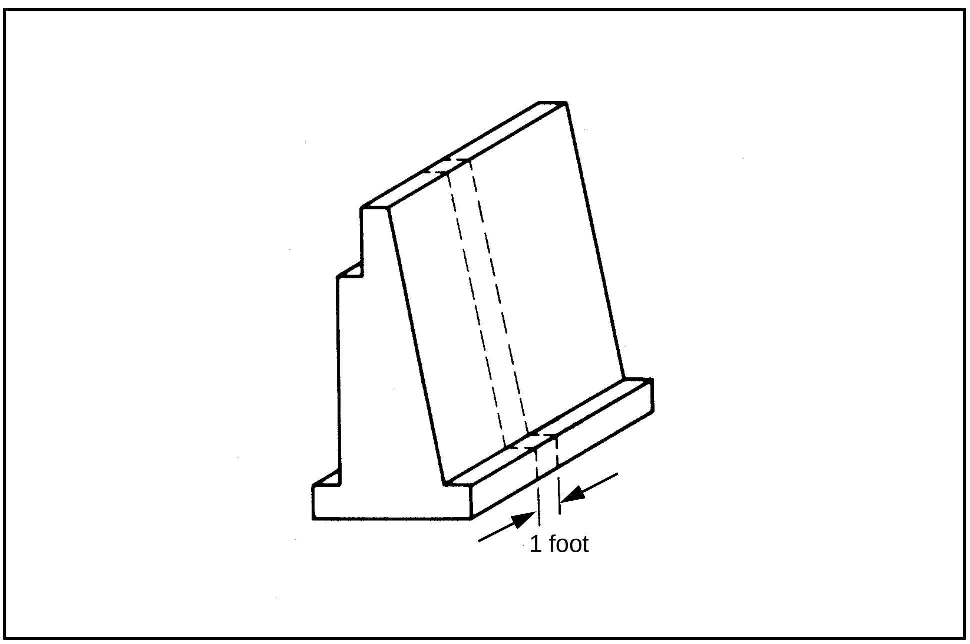


Figure 7-1. Typical Concrete Abutment

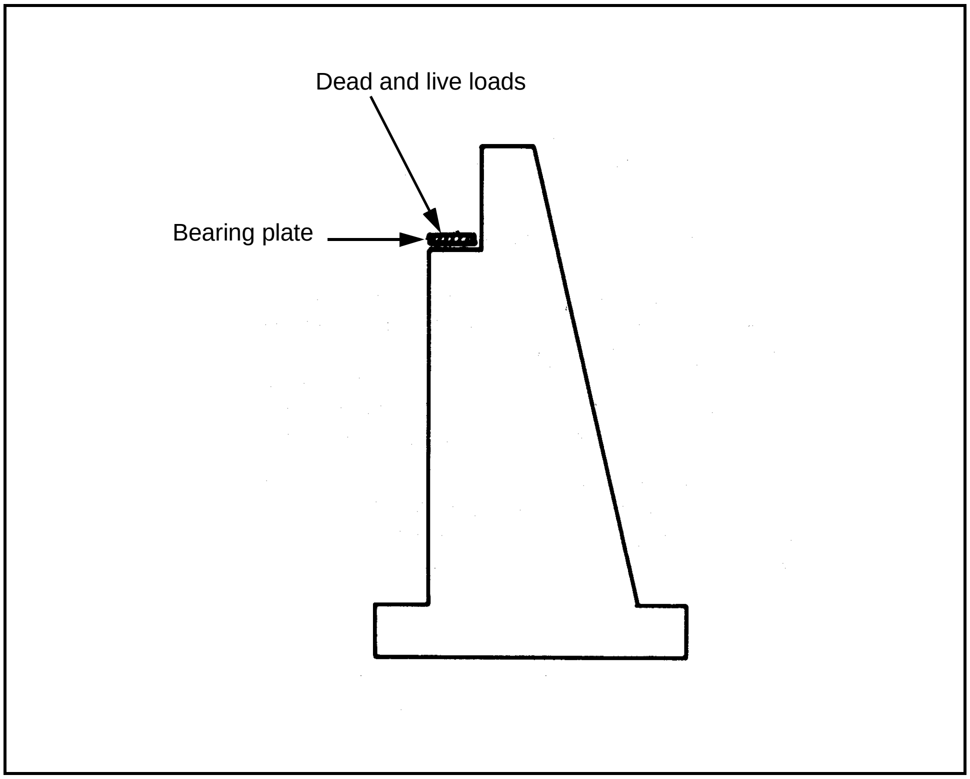


Figure 7-2. Dead- and Live-Load Action on an Abutment

Superstructure Dead Load

7-3. Compute the superstructure dead load as follows:

- **Piles or posts.**

$$P_{DL} = W'_{DL}\frac{L}{2} \qquad (7\text{-}1)$$

where—

P_{DL} = *dead load on the piles or posts, in kips*
W'_{DL} = *total actual dead load, in kpf (equation 6-43)*
L = *span length, in feet*

- **Concrete.**

$$P_{DL} = W'_{DL}\frac{L}{2L_a} \qquad (7\text{-}2)$$

where—

P_{DL} = *dead load on concrete, in kpf*
W'_{DL} = *total actual dead load, in kpf (equation 6-43)*
L = *span length, in feet*
L_a = *abutment length, in feet*

Live Load

7-4. The live load acts vertically through the centerline of the bearing plate *(Figure 7-2)*. Impact loads are not included. Compute the live load as follows:

- **Piles or posts.**

$$P_{LL} = V_{LL}N \tag{7-3}$$

where—

P_{LL} = *live load on piles or posts, in kips*
V_{LL} = *maximum live-load shear per lane, in kips (larger value from Figure B-3 or B-4, pages B-17 and B-18)*
N = *number of lanes*

- **Concrete.**

$$P_{LL} = \frac{V_{LL}N}{L_a} \tag{7-4}$$

where—

P_{LL} = *live load on concrete, in kpf*
V_{LL} = *maximum live-load shear per lane, in kips (larger value from Figure B-3 or B-4)*
N = *number of lanes*
L_a = *abutment length, in feet*

Abutment Weight

7-5. The weight of timber or steel abutments is negligible since it is small in comparison to other vertical abutment loads. However, the weight of concrete abutments should be included in the design loads. Divide the cross-sectional shape of an abutment into sections of known size, shape, and cross-sectional area *(Figure 7-3, page 7-4)*. The weight of any section acts vertically through the centroid of that section. Compute the weight of any section per foot of abutment length as follows:

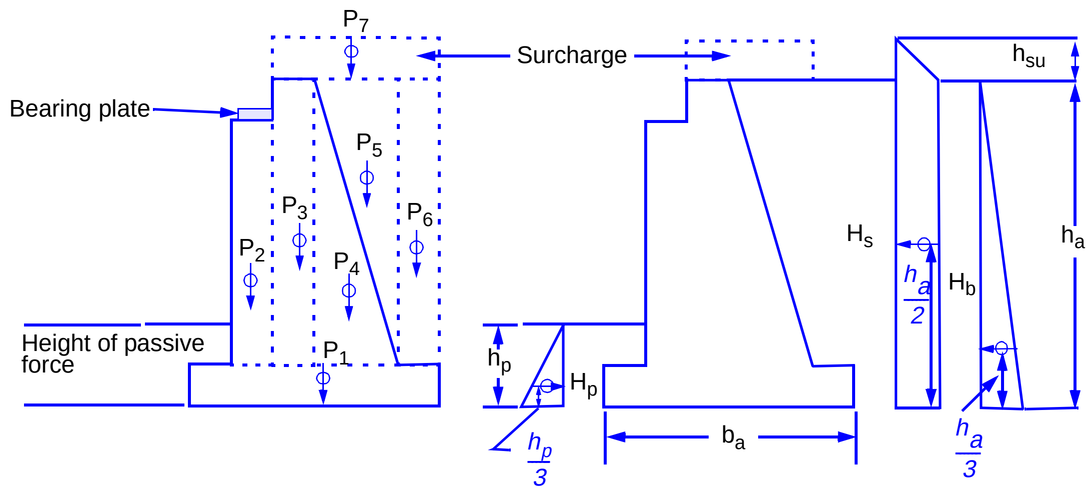
$$w = Au \tag{7-5}$$

where—

w = *weight of any abutment section (1-foot-wide strip), in kpf*
A = *area of the abutment section, in square feet*
u = *unit weight of abutment material (concrete weighs 0.15 kips per cubic foot)*

Soil Forces

7-6. Use *equation 7-5* and compute the weight of soil acting on the rear face and heel of the abutment (*Appendix I*). To account for vehicular traffic approaching the abutment, assume placing a hypothetical layer of soil



LEGEND:

b_a	= abutment width, in feet	h_p	= height of soil providing passive resistance
h_a	= abutment height	H_s	= horizontal force due to surcharge
H_b	= horizontal force due to active earth pressure	h_{su}	= height of surcharge
H_p	= horizontal force due to passive earth pressure	$P_{1\text{-}7}$	= vertical loads for Sections 1-7

Figure 7-3. Abutment Sectioning, Load Centers, and Soil Forces

(surcharge) over the backfill at the abutment. *Figure 7-3* shows vertical soil forces. Compute the height of the surcharge as follows:

$$h_{su} = \frac{MLC}{20} \qquad (7\text{-}6)$$

where—

h_{su} = *surcharge height, in feet*
MLC = *military load classification*

Hydrostatic Uplift

7-7. Properly locate the abutment, and provide drainage to avoid hydrostatic uplift forces on the abutment. Eliminate hydrostatic uplift forces by locating the abutment well above the flood stage, by providing granular material behind and under the abutment, and by installing weep holes in the abutment. Methods for draining water from behind and under abutments are discussed in *paragraphs 7-31* and *7-44*.

HORIZONTAL LOADS

7-8. Horizontal loads acting on abutments are created by soil, water, temperature, and vehicles. These loads are computed per foot of abutment length.

Soil Forces

7-9. Use the Rankine equations from *Introductory Soil Mechanics and Foundations: Geotechnic Engineering* to compute the horizontal surcharge and backfill forces as shown below. *Figure 7-3* shows the actions of these forces. In most cases, passive pressure is so negligible that it is not considered, resulting in a somewhat conservative design.

- **Coefficient of active earth pressure.**

$$K_a = tan^2\left[45° - \left(\frac{\theta}{2}\right)\right] \qquad (7\text{-}7)$$

where—

K_a = *coefficient of active earth pressure*
θ = *angle of internal friction (Table H-1, page H-1)*

- **Coefficient of passive earth pressure.**

$$K_p = tan^2\left[45° + \left(\frac{\theta}{2}\right)\right] \qquad (7\text{-}8)$$

where—

K_p = *coefficient of passive earth pressure*
θ = *angle of internal friction (Table H-1)*

- **Soil forces and point of application.**
  - **Horizontal surcharge force.**

$$H_s = (K_a u h_{su})h_a \text{ @ } \frac{h_a}{2} \qquad (7\text{-}9)$$

where—

H_s = *horizontal surcharge force, in kpf*
K_a = *coefficient of active earth pressure (equation 7-7)*
u = *unit weight of backfill, in kips per cubic foot (Table H-1)*
h_{su} = *surcharge height, in feet (equation 7-6)*
h_a = *abutment height, in feet*

  - **Horizontal backfill force.**

$$H_b = (K_a u h_a)\left(\frac{h_a}{2}\right) \text{ @ } \frac{h_a}{3} \qquad (7\text{-}10)$$

where—

H_b = *horizontal backfill force, in kpf*
K_a = *coefficient of active earth pressure (equation 7-7)*
u = *unit weight of backfill, in kips per cubic foot (Table H-1)*
h_a = *abutment height, in feet*

- **Horizontal force due to passive earth pressure.**

$$H_p = K_p u h_p \left(\frac{h_p}{2}\right) \qquad (7\text{-}11)$$

where—

H_p = *horizontal force due to passive earth pressure, in kpf*
K_p = *coefficient of passive earth pressure (equation 7-8)*
u = *unit weight of backfill, in kips per cubic foot (Table H-1, page H-1)*
h_p = *height of passive force, in feet (Figure 7-3, page 7-4)*

Water Forces

7-10. Water that collects behind an abutment will create a horizontal force that can be eliminated by proper drainage. See *paragraph 7-45* for methods of draining water from behind and under abutments.

Temperature Forces

7-11. Temperature forces are negligible in simply supported stringer bridges designed for military purposes. Bearing plates support each end of every steel stringer. One end of the stringer is rigidly attached so that there is no movement between the stringer and the support. The other end is attached so that movement is allowed along the length of the stringer. As the stringer expands or contracts due to temperature changes, temperature forces are dissipated at the free end. The fixed bearing plate is normally placed on the abutment end of the stringer.

Vehicular Longitudinal Forces

7-12. As vehicles accelerate or brake on a bridge span, a longitudinal force is transmitted to the substructure through the fixed bearing plates. Compute the vehicular force as follows:

$$H_v = \frac{0.05WN}{L_a} \qquad (7\text{-}12)$$

where—

H_v = *vehicle force per foot of abutment length, in kips*
W = *vehicle weight, in kips*
N = *number of vehicles per lane*
L_a = *abutment length, in feet*

7-13. The number of vehicles allowed per lane is based on the span length and the vehicle spacing (at least 100 feet). Since vehicles traveling in opposite directions have canceling effects on loads, the worst loading conditions occur when vehicles move in only one direction. Therefore, the number of vehicles on a span is for one lane only. At the expansion bearing plate, the vehicle force is zero. The vehicle force acts horizontally on the fixed bearing plate *(Figure 7-4).*

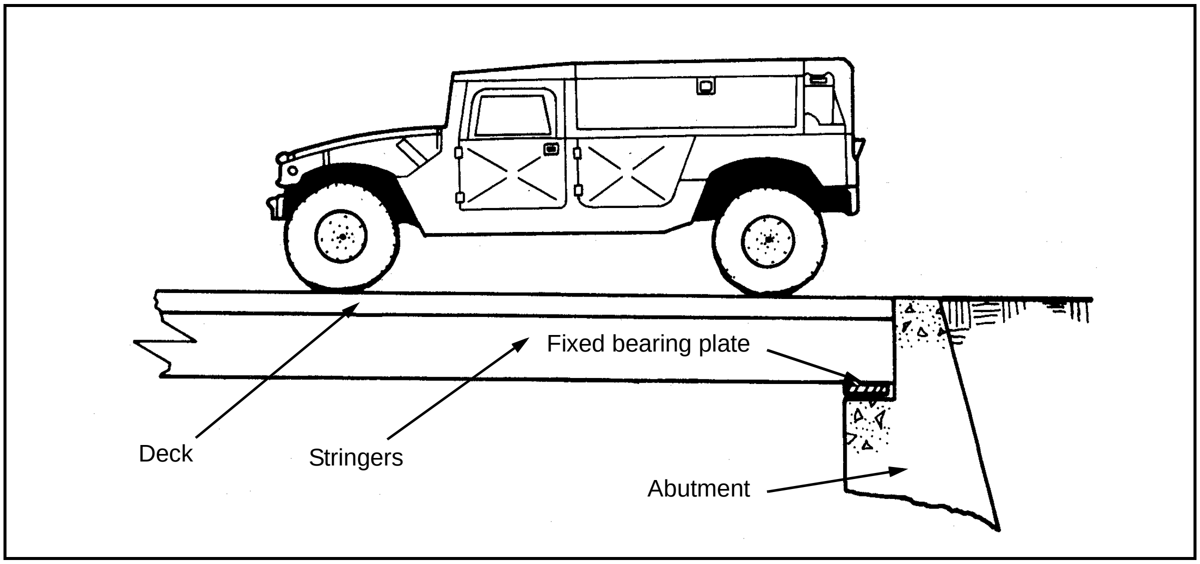


Figure 7-4. Vehicular Longitudinal Force

LATERAL LOADS

7-14. Lateral loads acting on abutments are negligible.

ABUTMENT SELECTION

7-15. Abutments may be timber, steel, concrete, or a combination of these materials. The abutments may rest directly on the soil (as in the case of concrete), or they may rest on footings or piles. *Figure 7-5* and *Figures 7-6* through *7-11, pages 7-8* through *7-10,* show typical abutments. *Table 7-1, page 7-11,* gives a general guide to the types of abutments used under various conditions.

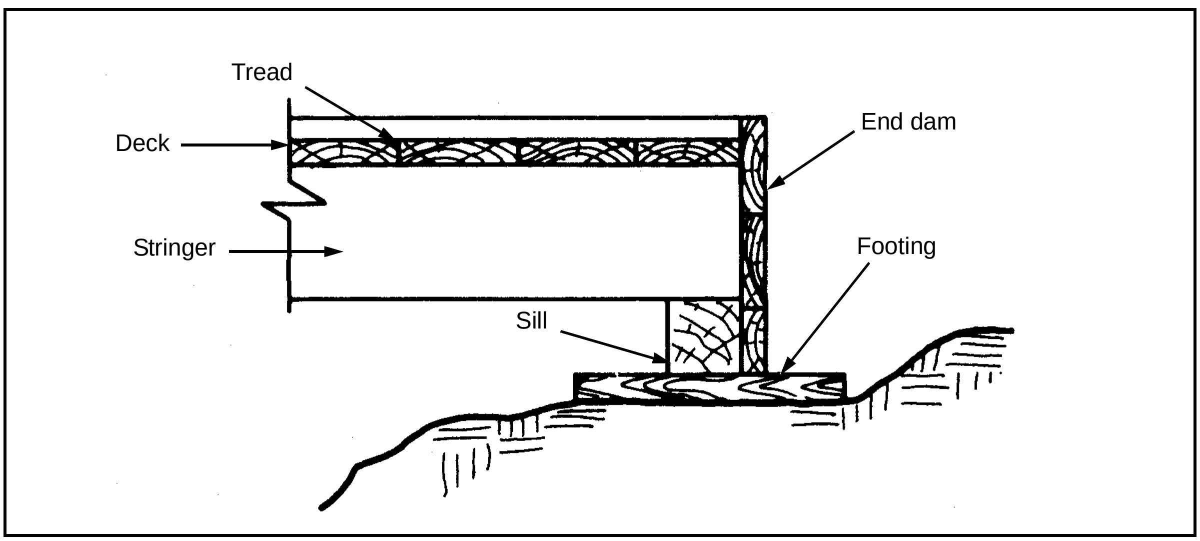


Figure 7-5. Timber-Sill Abutment

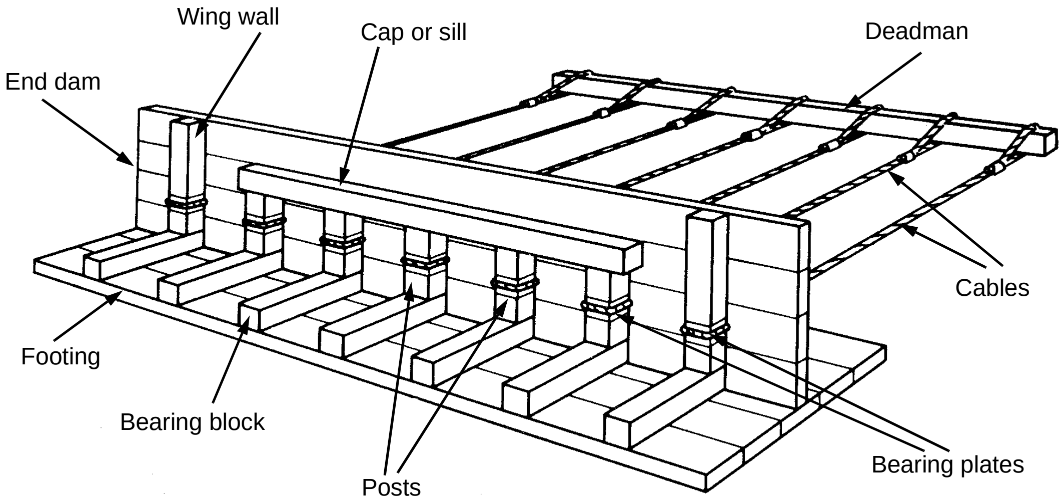


Figure 7-6. Timber-Bent Abutment

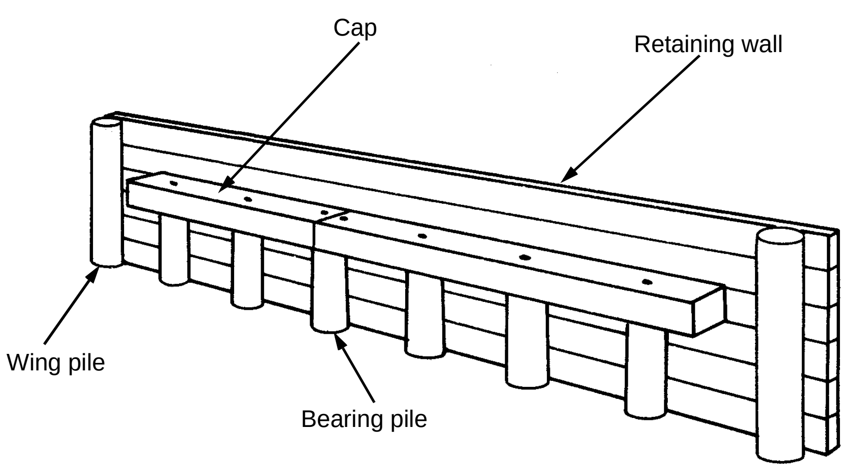


Figure 7-7. Timber-Pile Abutment

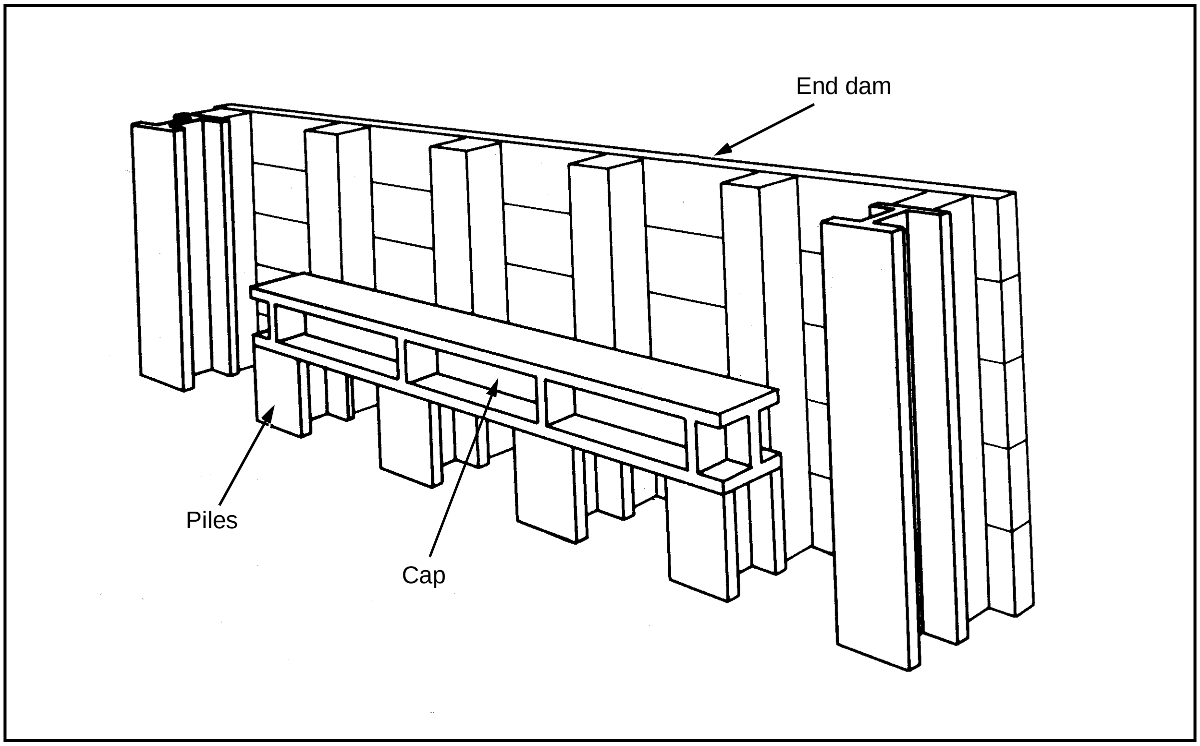


Figure 7-8. Steel-Pile Abutment

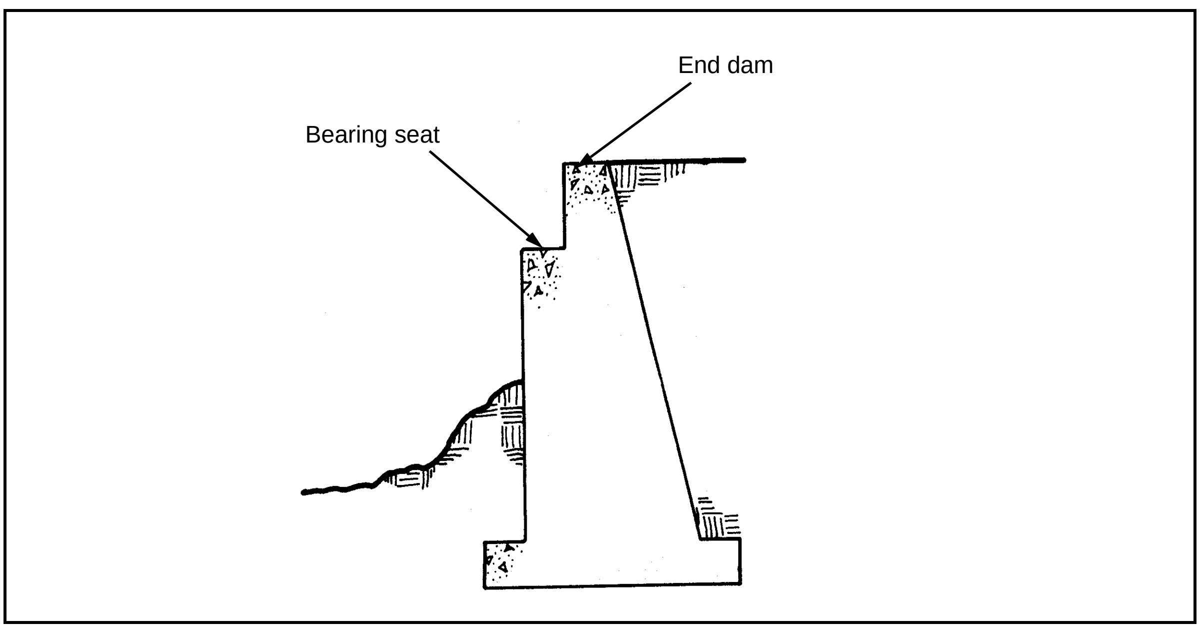


Figure 7-9. Mass-Concrete Abutment

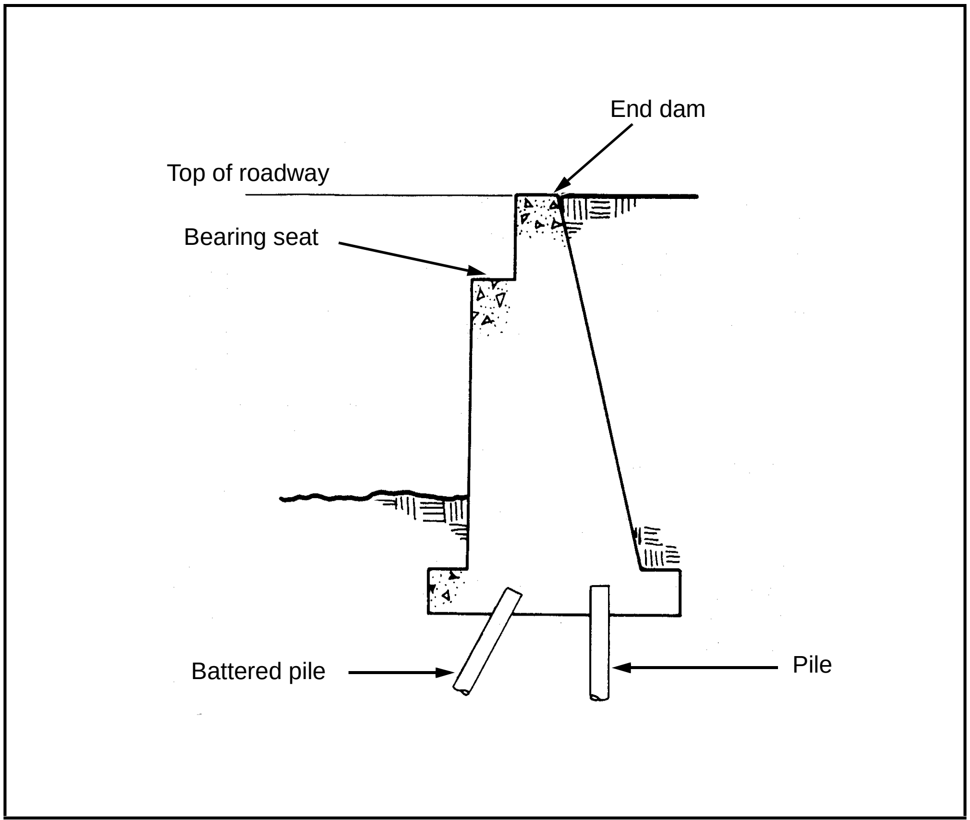


Figure 7-10. Concrete Abutment on Piles

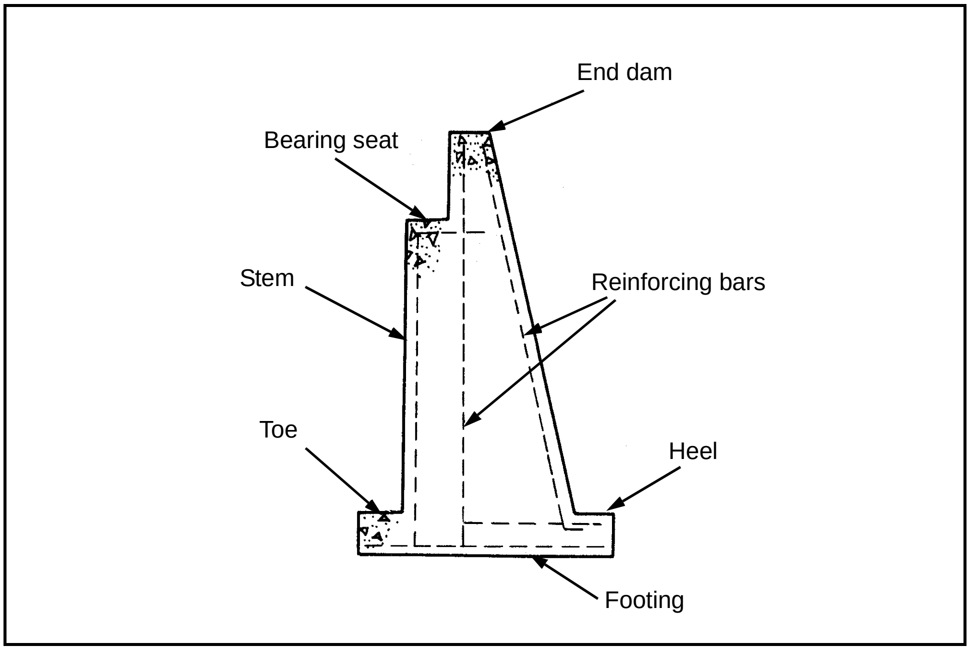


Figure 7-11. Reinforced Concrete Abutment

Table 7-1. Abutment Types

Type	Span	Height	Remarks
Timber sill	≤ 25 feet	≤ 3 feet	Highway bridges only. Designed for vertical loads only and steel or timber stringers.
Timber bent	≤ 30 feet	≤ 6 feet	High bridges only. Designed for vertical loads. Deadman used for horizontal stability. Used with steel or timber stringers.
Timber or steel pile	Any length	≤ 10 feet	Designed for vertical and longitudinal loads and steel or timber stringers.
Mass or reinforced concrete	Any length	≤ 20 feet	Most permanent type. Designed for vertical and longitudinal loads and steel or timber stringers.

7-16. The general principles of abutment design are applicable to any type of abutment resting on the ground or on footings, with only a few modifications for abutments resting on piles. These principles also apply directly to the design of retaining walls. Abutments must be designed to avoid the following types of failure:

- Overturning.
- Sliding.
- Soil bearing.
- Soil shear.
- Material.

7-17. Since abutments are subjected to various types of loadings during construction and use, the load cases must be considered *(Table 7-2)*. *Figure 7-12, page 7-12,* shows a sample chart of abutment loads and moments.

Table 7-2. Load Cases

Case	Description	Vertical Loads	Horizontal Loads
I	Unloaded	Abutment weight plus the soil weight (including the surcharge)	Soil forces (including the surcharge)
II	Dead load	Case I plus the superstructure's dead load	Case I
III	Live load	Case II plus the live load minus the surcharge	Case II plus the vehicular longitudinal force minus the surcharge

Case	Load (kips)	Arm (ft)	M_r (kip-ft)	Load (kips)	Arm (ft)	M_o (kip-ft)
—	$P_1 = 1.425$	2.375	3.384	$H_s = 0.902$	4.000	3.608
—	$P_2 = 0.438$	1.500	0.657	$H_b = 1.202$	2.667	3.206
—	$P_3 = 0.900$	2.500	2.250	—	—	—
—	$P_4 = 0.450$	3.333	1.500	—	—	—
—	$P_5 = 0.390$	3.667	1.430	—	—	—
—	$P_6 = 0.585$	4.375	2.559	—	—	—
—	$P_7 = 1.073$	3.375	3.621	—	—	—
I	$\sum P = 5.261$ $P_{DL} = 0.821$	1.500	$\sum M_r = 15.401$ 1.232	$\sum H = 2.104$	—	$\sum M_o = 6.814$
II	$\sum P = 6.082$ $P_{LL} = 6.171$	1.500	$\sum M_r = 16.633$ 9.527	$\sum H = 2.104$ $H_v = 0.250$	4.920	$\sum M_o = 6.814$ 1.230
	$-P_7 = 1.073$	3.375	-3.621	$-H_s = 0.902$	4.000	-3.608
III	$\sum P = 11.180$	—	$\sum M_r = 22.269$	$\sum H = 1.452$	—	$\sum M_o = 4.436$

LEGEND:

H = horizontal load
H_b = horizontal backfill force, in kpf
H_s = horizontal surcharge force, in kpf
H_v = vehicle force per foot of abutment length, in kips
M_o = overturning moment, in kpf
M_r = minimum resisting moment per pile, in kip-feet
P = total load, in kips
P_{1-7} = vertical loads for Sections 1-7
P_{DL} = dead load of the substructure, in kips
P_{LL} = live load of the substructure, in kips

Figure 7-12. Sample Chart of Abutment Loads and Moments

ABUTMENT DESIGNS

TIMBER-FOOTING AND TIMBER-BENT ABUTMENTS

7-18. Determine the loads using the procedures in *paragraphs 7-2* through *7-4*. Consider only live and dead loads. Compute the total load for the abutment as follows:

$$P = P_{DL} + P_{LL}\left(\frac{N_s}{4}\right) \tag{7-13}$$

where—

P = *total load, in kips*
P_{DL} = *dead load of the substructure, in kips (equation 7-1 or 7-2)*
P_{LL} = *live load of the substructure, in kips (equation 7-3 or 7-4)*
N_s = *number of stringers supported by the abutment (equation 6-1)*

Abutments

7-19. Choose the abutment length so that the following requirement is met:

$$b_R < L_a \leq (b_R + 4\ ft) \tag{7-14}$$

where—

b_R = *curb-to-curb roadway width, in feet*
L_a = *abutment length, in feet*

Posts

7-20. Compute the allowable bearing capacity per post and the required number of posts as follows:

- **Allowable bearing capacity per post.**

$$P_B = F_c A \tag{7-15}$$

where—

P_B = *allowable bearing capacity of the pile or post, in kips*
F_c = *allowable compression, parallel to the grain of the post material, in ksi (Table C-1, pages C-3 through C-6)*
A = *cross-sectional area of the post, in square inches (Table C-5, page C-10)*

- **Required number of posts.** The number of posts must be greater than or equal to the number of stringers.

$$N_p = \frac{P}{P_B} \tag{7-16}$$

where—

N_p = *number of posts in the abutment (the number of posts is greater than or equal to the number of stringers*
P = *total load, in kips (equation 7-13)*
P_B = *allowable bearing capacity of the post, in kips (equation 7-15)*

Caps and Sills

7-21. The following equations apply to both caps and sills. The absolute minimum size for a cap or sill is 6 x 8 inches. If a sill is used, it should be the same size as the cap.

- **Width.** After determining the cap or sill width for stringers, substitute the post size for the stringer size in the following equation and use the larger of the two results as the cap or sill width:

$$b_c = \frac{P}{N_p b_s F_c} \tag{7-17}$$

where—

b_c = *cap or sill width, in inches*
P = *total load, in kips (equation 7-13)*
N_p = *number of posts in the abutment (equation 7-16 or DA Form 1249)*
b_s = *stringer or post width, in inches*
F_c = *allowable compression of the supports perpendicular to the grain of the support material, in ksi (Table C-1, pages C-3 through C-6)*

- **Depth.** Compute the depth as follows:

$$d = \frac{L_a 12}{(N_p - 1)5} \tag{7-18}$$

where—

d = *cap depth, in inches*
L_a = *abutment length, in feet*
N_p = *number of posts (equation 7-16)*

Footings

7-22. Each post or stringer (whichever is used) must rest on a footing. Compute the allowable length, the area, the capacity, and the number of footings as follows:

- **Allowable length.**

$$L_f = b_c + K \tag{7-19}$$

where—

L_f = *footing length, in inches. (Round the footing length down if desired.)*
b_c = *cap or sill width, in inches (equation 7-17)*
K = *soil-bearing-capacity coefficient (Figure H-1, page H-3)*

- **Area.**

$$A_f = L_f b_f \tag{7-20}$$

where—

A_f = *footing area, in square inches*
L_f = *actual footing length, in inches (equation 7-19)*
b_f = *actual footing width, in inches*

- **Capacity.**

$$F_f = A_f F_{Bs} \tag{7-21}$$

where—

F_f = *footing capacity, in kips*

A_f = *footing area, in square feet (equation 7-20)*
F_{Bs} = *soil bearing capacity, in kips per square foot (ksf) (Table H-1)*

- **Number of footings.** The required number of footings will be greater than or equal to the number of posts or stringers.

$$N_f = \frac{P}{F_f} \qquad (7\text{-}22)$$

where—

N_f = *number of footings*
P = *total load, in kips (equation 7-13)*
F_f = *footing capacity, in kips (equation 7-21)*

End Dam and Deadmen

7-23. Material used in an end dam should be at least 3 inches thick to prevent failure from earth pressure. Install deadmen if using a timber bent (see *paragraph 7-58* for a deadman design).

TIMBER- AND STEEL-PILE ABUTMENT DESIGNS

7-24. Follow the procedures described for the design of pile foundations in *paragraph 7-100*.

MASS-CONCRETE ABUTMENT DESIGN

7-25. Mass-concrete abutments are used when the abutment will or is expected to be in contact with the stream or if the required height is greater than ten feet. Design mass-concrete abutments by trial and error.

Dimensions

7-26. Base the estimate of the overall height of an abutment on local site conditions. An abutment should extend far enough below the ground's surface so that it rests on firm soil. If the overall height exceeds 20 feet, a mass-concrete structure becomes uneconomical and a pile foundation should be considered. Once the abutment height is determined, use the following guidelines to estimate other dimensions *(Figure 7-13, page 7-16)*. Always check the safety of any abutment designed by these guidelines. Assume the following dimensions and loads:

- The abutment width is 40 to 60 percent of the abutment height, in feet.
- The footing height is 1 to 2 feet.
- The toe and heel width are ¾ to 1 ½ feet and are less than or equal to 75 percent of the footing height.
- The seat width is greater than or equal to the cap or sill width, and the cap or sill width is greater than or equal to ¾ foot.
- The superstructure height is measured from the bottom of the bearing plate to the top of the deck, in feet.

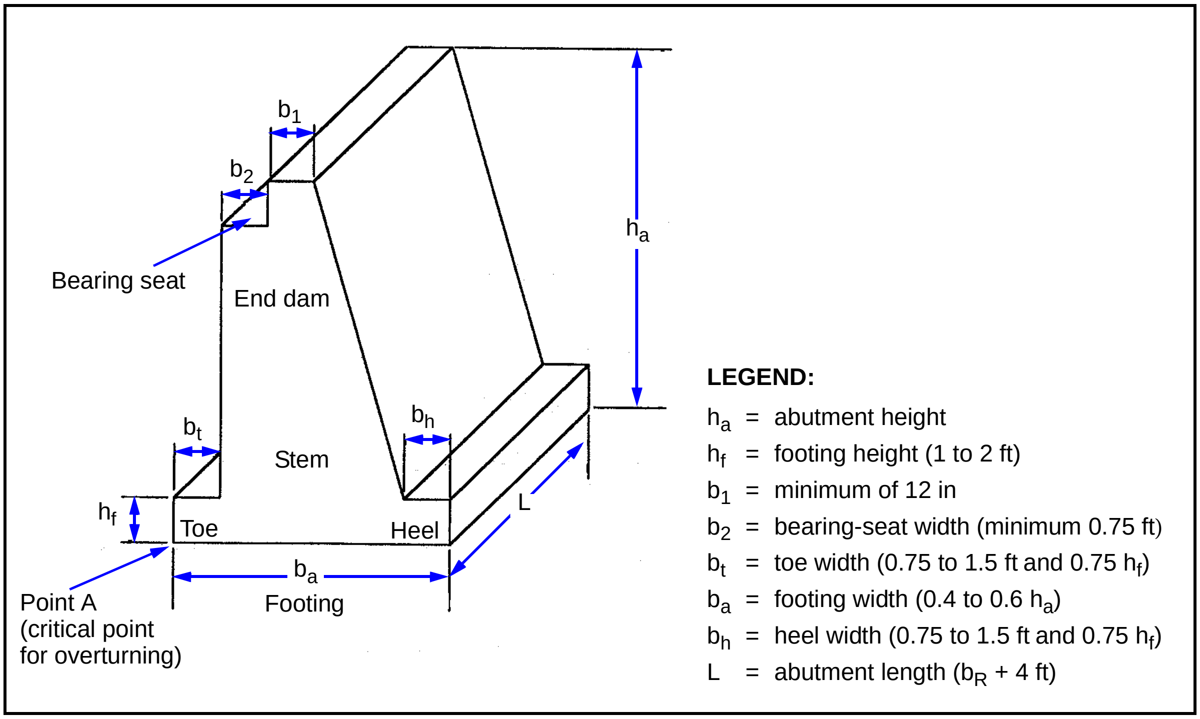


Figure 7-13. Mass-Concrete-Abutment Design Dimensions

7-27. If any of the safety criteria fail, choose new dimensions. A heel and toe are not mandatory for mass-concrete abutments, but they are recommended because they provide additional stability and aid in drainage. Once the preliminary dimensions are selected, determine all the loads that act on the abutment, using the methods described in *paragraphs 7-2* and *7-8*.

Loading Case Values

7-28. For easy computations, set the values for the loads, lever arms, and moments as shown in *Figure 7-12, page 7-12*. The moment arm is the perpendicular distance from the abutment toe to the line of action of the given load. Determine the moments about the toe, since this is the critical point for overturning. Use the tabulated values to check the safety of the abutment against sliding, overturning, and soil-bearing failure.

Sliding Check

7-29. Compute the safety factor of sliding for loading Cases I and III as follows:

$$SF = \frac{\Sigma P(K_f)}{\Sigma H} \geq 1.5 \tag{7-23}$$

where—

SF = *safety factor (if the safety factor is less than 1.5 for any loading case, take steps to prevent sliding [as discussed below])*

ΣP = *total of the vertical loads, in kips (paragraph 7-2)*

K_f = *friction coefficient, (Table H-2, page H-2)*
ΣH = *total of the horizontal forces, in kips (paragraph 7-8)*

Overturning Check

7-30. Compute the safety factor of overturning for loading Cases I and III as follows:

$$SF\frac{\Sigma M_r}{\Sigma M_o} \geq 2 \qquad (7\text{-}24)$$

where—

SF = *safety factor for overturning (if the safety factor is less than 2, take steps to prevent overturning [as discussed below])*
ΣM_r = *total of the resisting moments, in kip-feet (Figure 7-12)*
ΣM_o = *total of the overturning moments, in kip-feet (Figure 7-12)*

Sliding and Overturning Prevention

7-31. Do one or more of the actions discussed below to prevent sliding and overturning.

7-32. **Modify the Abutment or Heel.** Increase the size of the abutment or the length of the heel so that more weight acts vertically.

7-33. **Modify the Superstructure Placement.** If sliding or overturning is critical for Case I (unloaded) only, place the superstructure onto the abutment before backfilling the space behind the abutment.

7-34. **Modify the Friction Coefficient.** Increase the friction coefficient by adding a layer of gravel under the abutment. Use a filter layer to prevent fine material from washing out through the gravel.

7-35. **Install a Deadmen.** Install a deadmen behind the abutment. *Paragraph 7-58* describes a deadman design.

7-36. **Construct a Key.** Construct a key as shown in *Figure 7-14, page 7-18*. This key will develop passive earth forces that will help resist sliding. Use half the value of the horizontal force in safety computations, since the full value will probably never be developed.

- **Compute the horizontal force due to passive earth pressure.** Compute as follows:

$$H_f = \frac{K_p u(h_1^2 - h_2^2)}{2} \qquad (7\text{-}25)$$

where—

H_f = *horizontal force due to the passive earth pressure, per foot of abutment, in kips*
K_p = *coefficient of passive earth pressure (equation 7-8)*
u = *unit weight of backfill, in kips per cubic foot (Table H-1, page H-1)*
h_1 = *passive force height, including the key, in feet (Figure 7-14)*
h_2 = *original passive force height, in feet (Figure 7-14)*

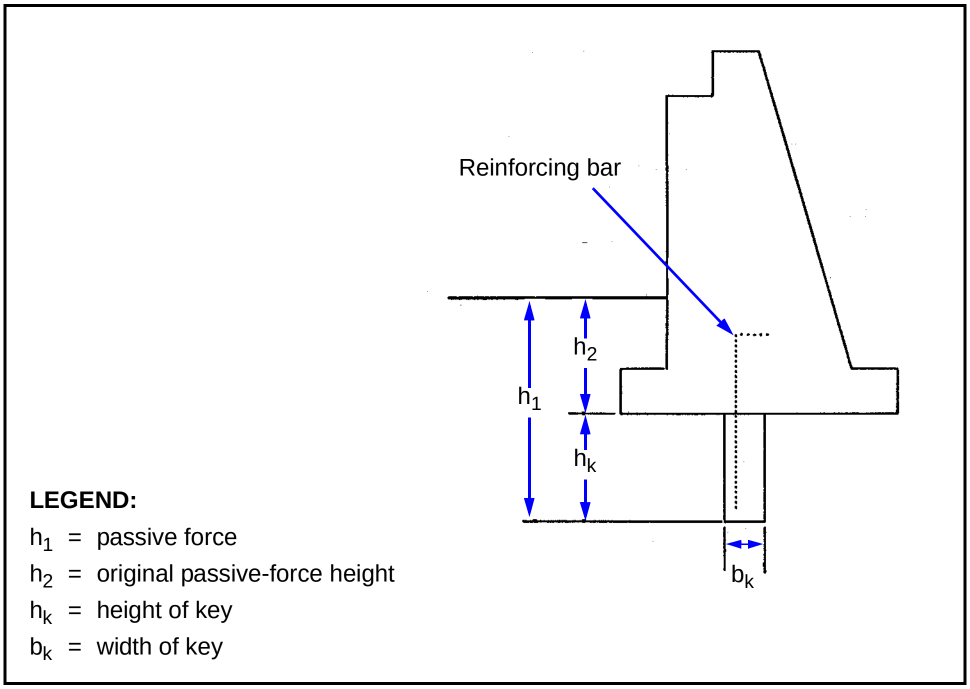


Figure 7-14. Abutment Key

- **Compute the dimensions.** If necessary, choose and modify the preliminary key dimensions as follows:

$$h_k \approx \frac{h_a}{4} \tag{7-26}$$

where—

h_k = *key height, in feet*
h_a = *abutment height, in feet*

and—

$$b_k \approx \frac{h_k}{3} \tag{7-27}$$

where—

b_k = *key width, in feet (greater than or equal to 1 foot)*
h_k = *key height, in feet*

- **Reinforce the key to prevent failure due to concrete stresses.** Use No. 4 bars with 1-foot, center-to-center spacing. Extend the bars 1 foot into the body of the abutment. Use a concrete cover (minimum of 3 inches) over the reinforcing steel.

- **Modify the design.** If placing the abutment on piles, design the piles to carry the full load (including the abutment weight). *Paragraph 7-100* describes pile foundations.

Soil Bearing Capacity

7-37. Determine the allowable soil bearing capacity for the specific site, if possible. Otherwise, use *Appendix H* to estimate the soil bearing capacity. The criteria for proper soil bearing capacity are the—

- Eccentric distance is less than or equal to one-sixth the abutment width for all three cases.
- Minimum pressure of eccentricity is less than or equal to the soil bearing capacity for Case III.

7-38. **Eccentric Distance.** Since horizontal and vertical forces act on an abutment, the pressure it exerts on the soil varies. The resultant force due to soil pressure acts at an eccentric distance from the geometric center of the abutment.

7-39. Compute the eccentric distance as follows:

$$e = \frac{b_a}{2} - \frac{\Sigma M_r - \Sigma M_o}{\Sigma p} \qquad (7\text{-}28)$$

where—

e = *eccentric distance, in feet*

b_a = *abutment width, in feet*

ΣM_r = *total of all resisting moments for the specific case, in kip-feet (Figure 7-12, page 7-12)*

ΣM_o = *total of all overturning moments for a specific case, in kip-feet (Figure 7-12)*

ΣP = *total of the vertical forces acting downward for a specific case, in kips (Figure 7-12)*

7-40. If the sign of the eccentric distance is positive, the resultant force is to the left of the centerline and maximum pressure occurs at the toe *(Figure 7-15A, page 7-20).* If the sign of the eccentric distance is negative, the resultant force is to the right of the centerline and maximum pressure occurs at the heel *(Figure 7-15B).*

7-41. **Load Eccentricity.** The resultant force must be within the middle third of the abutment. If the eccentric distance is less than one-sixth of the abutment width, the resultant force is within the middle third of the abutment. If the eccentric distance is greater than one-sixth of the abutment width, the resultant force is outside the middle third, indicating that the soil is in tension *(Figure 7-16, page 7-20).* However, since soil cannot take tension, the resultant load is spread over a smaller area, which increases the maximum pressure and possibly leads to failure. Determine the eccentricity of load for each loading case to ensure that the eccentric distance is less than or equal to one-sixth of the abutment width. To correct excessive load eccentricity, increase the toe or heel length so that the eccentric distance is within the middle third of the abutment.

7-42. **Maximum Soil Pressure.** The maximum soil pressure must not exceed the allowable soil bearing capacity. Determine the maximum pressure for Case III only, since the worst loading condition occurs for this case.

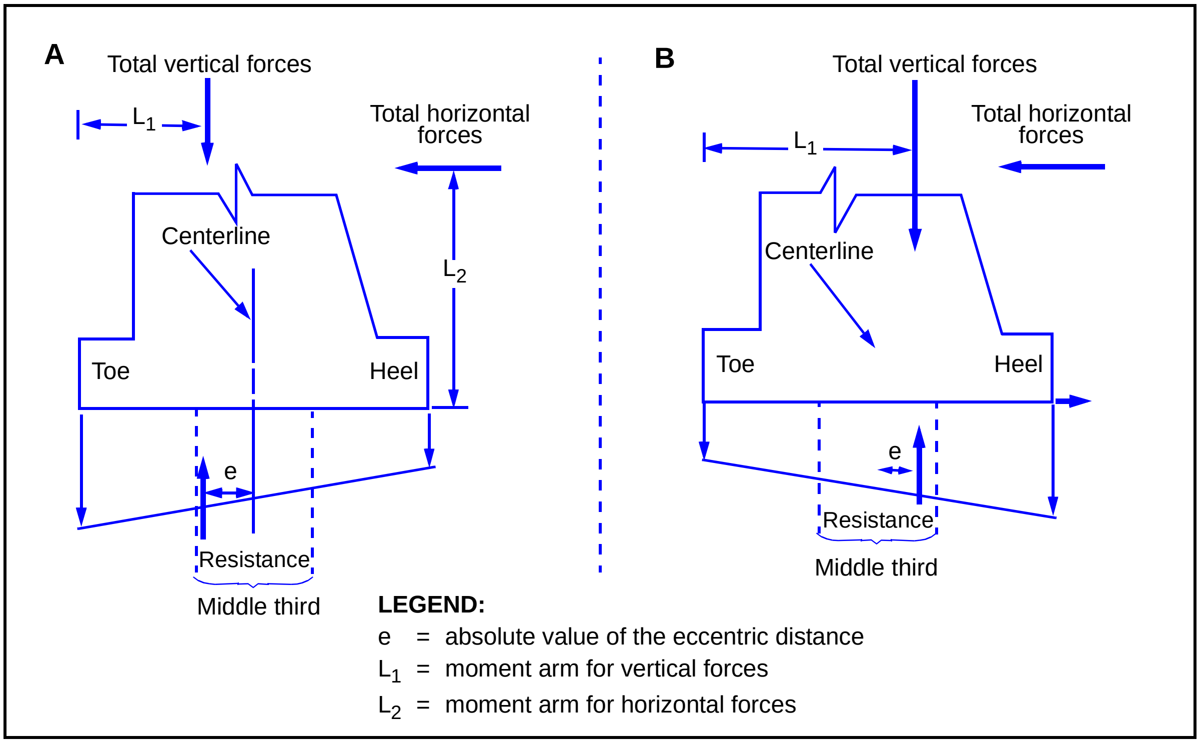


Figure 7-15. Eccentric Distance

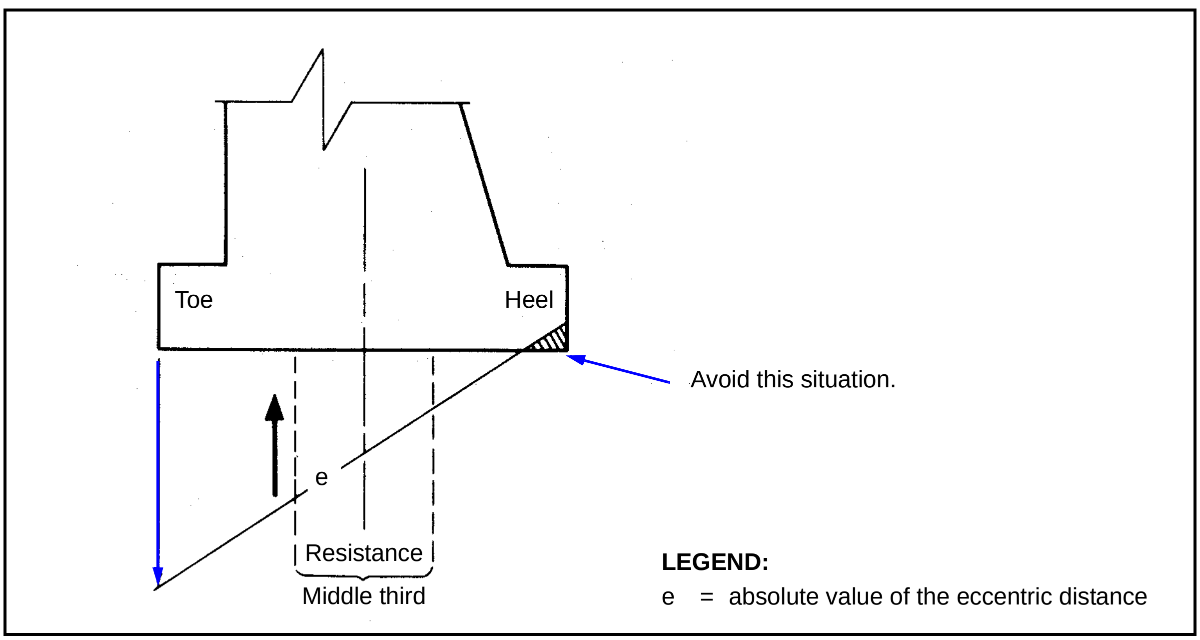


Figure 7-16. Eccentricity Outside the Middle Third of the Abutment

Compute the maximum pressure for eccentricity within the middle third of the abutment as follows:

$$P_{mx} = \frac{\Sigma P}{b_a}\left(1 + \frac{6_e}{b_a}\right) \tag{7-29}$$

where—

P_{mx} = *maximum pressure exerted on the soil, in ksf*
ΣP = *total of the vertical loads, in kips (paragraph 7-2)*
e = *absolute value of the eccentric distance, in feet (equation 7-28)*
b_a = *abutment width, in feet*

7-43. **Minimum Soil Pressure.** Compute the minimum soil pressure for eccentricity within the middle third of the abutment as follows:

$$P_{mn} = \frac{\Sigma P}{b_a}\left(1 - \frac{6_e}{b_a}\right) \tag{7-30}$$

where—

P_{mn} = *minimum pressure exerted on the soil, in ksf*
ΣP = *total of the vertical loads, in kips (paragraph 7-2)*
e = *absolute value of the eccentric distance, in feet (equation 7-28)*
b_a = *abutment width, in feet*

Soil Shear Failure

7-44. Soil shear failure is a landslide-type failure involving the abutment and the mass of earth under and behind the abutment. *Figure 7-17* shows typical soil shear failure. The primary causes of soil shear failure are—

- The slope in front of the abutment is too steep.
- Scour occurs under the abutment.
- The backfill is saturated with water or eroded from the sides of the abutment.

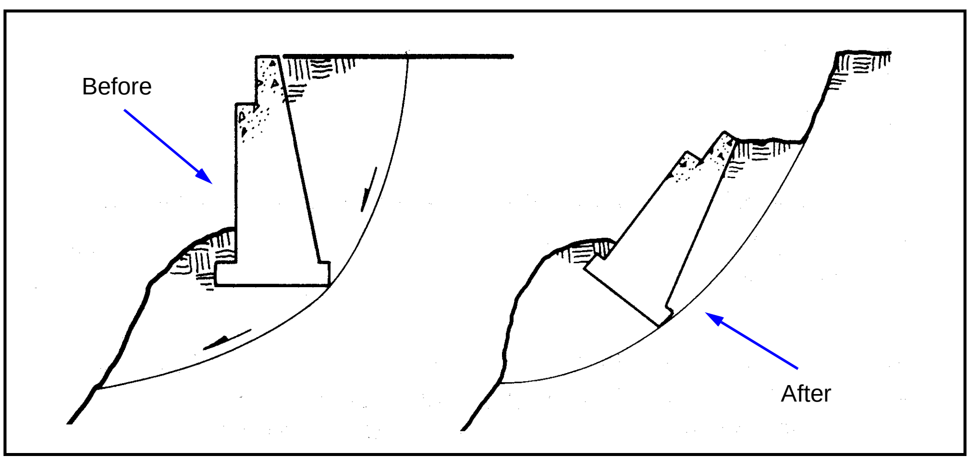


Figure 7-17. Soil Shear Failure

7-45. To prevent soil shear failure, ensure that the slope in front of the abutment is no more than 45 degrees and preferably less than 35 degrees *(Figure 7-18A)*. Place riprap in front of the abutment to prevent scour. If riprap is not available or the slope cannot be made less than 45 degrees, drive sheet piles in front of the abutment. The sheet piles must extend below the bottom of the gap and be anchored near the top with a deadman *(Figure 7-18B)*. The abutment can be placed on friction or bearing piles. Backfill the abutment with sand and gravel and install weep holes for drainage. Protect weep holes with a filter. Also, install granular material under the abutment, especially if the soil contains clay. Install wing walls to prevent erosion of the earth from the sides of the abutment *(Figure 7-18C)*.

Concrete Failure

7-46. *Figure 7-19, page 7-24,* shows the critical sections in a mass-concrete abutment. Section A-A is especially critical because Point C is subject to vehicle impact loads if the backfill settles. To prevent failure, reinforce Section A-A with steel bars *(Figure 7-20, page 7-24)*. See *Figure 7-21, page 7-25,* for concrete-stress checkpoints.

REINFORCED CONCRETE ABUTMENT

7-47. Use reinforced concrete abutments if economy of materials is desired. Concrete abutments are subject to the same types of failure as other abutments. Preventing failure (except for concrete failure) requires the same procedures as for mass-concrete abutments. *Figure 7-22, page 7-25,* shows a typical reinforced concrete abutment. Check Sections A-A, B-B, D-D, and E-E for potential failure. Follow the procedures discussed below when designing reinforced concrete abutments.

Dimensions

7-48. Local soil conditions govern the abutment height. Choose a height that allows the footing to rest on firm soil. Choose the other dimensions using the following limits (check all critical sections for safety after choosing the dimensions):

- The curb-to-curb roadway width is less than the abutment length, and the abutment length is less than the roadway width plus 4 feet.
- The abutment width is 50 to 70 percent of the abutment height.
- The stem width is 15 percent of the abutment height.
- The toe and heel heights are 10 percent of the abutment height.
- The seat width is greater than or equal to the cap or sill width, and the cap or sill width is greater than or equal to ¾ foot.
- The superstructure depth is measured from the bottom of the bearing plate to the top of the deck.
- The toe width is 10 percent of the abutment height.

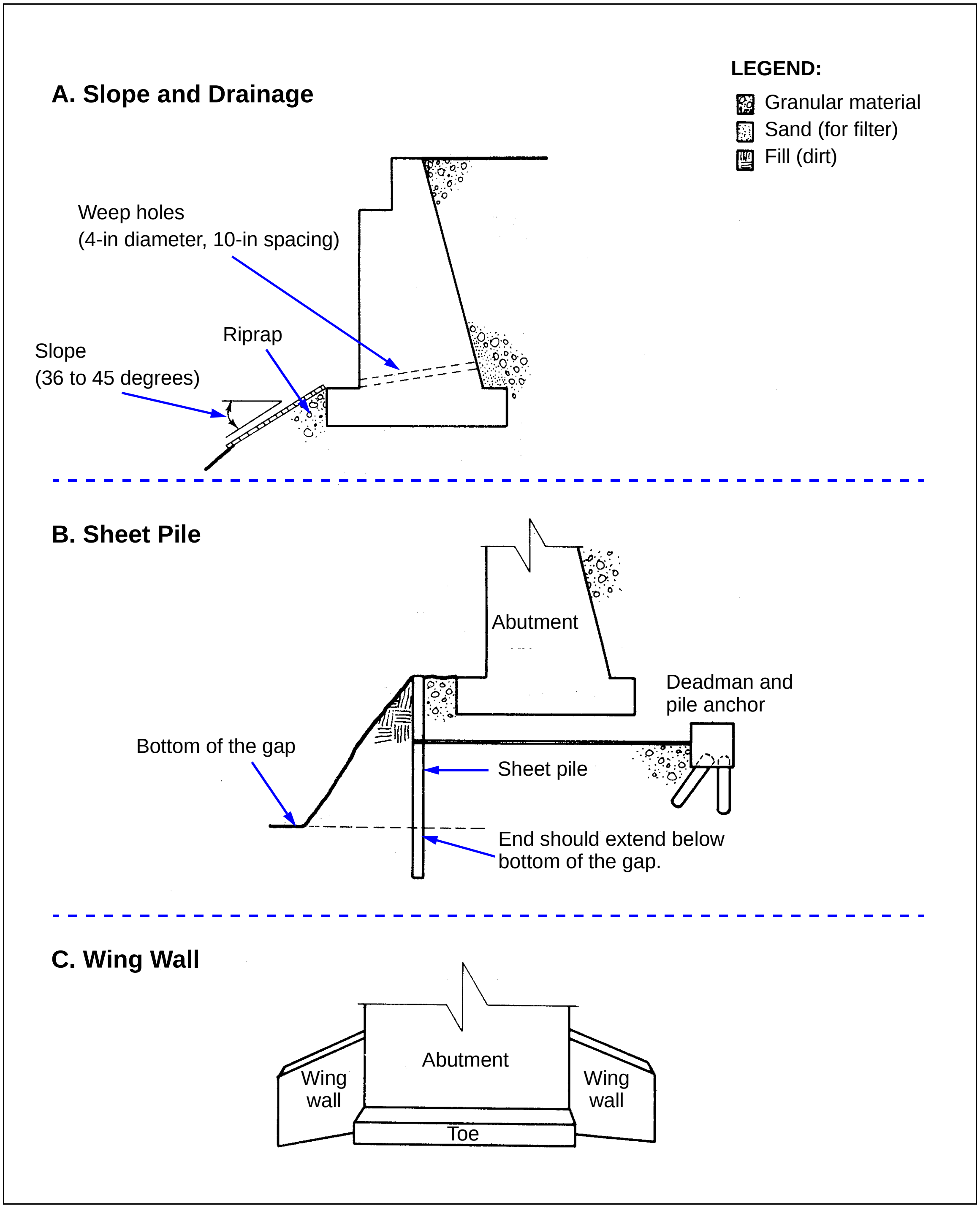


Figure 7-18. Preventing Soil Shear Failure

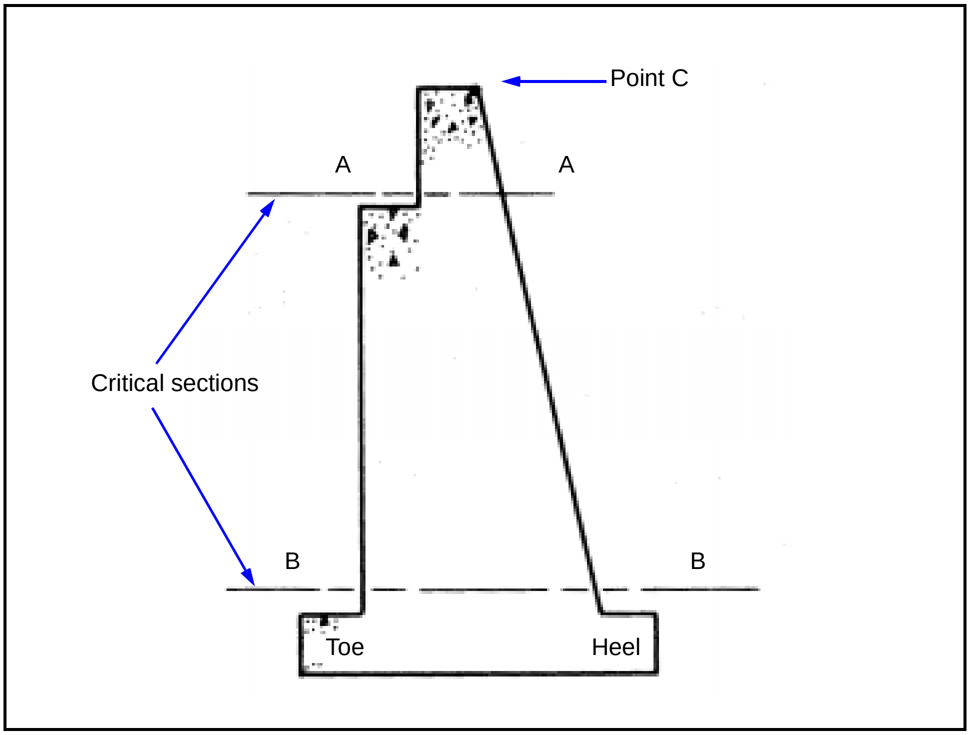


Figure 7-19. Critical Sections in a Mass-Concrete Abutment

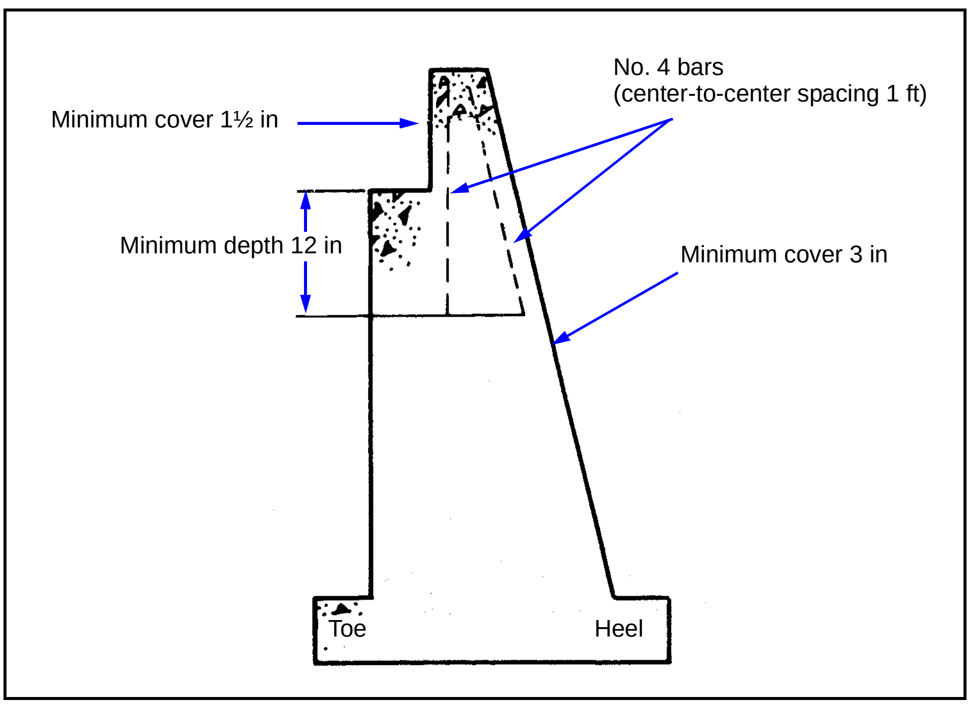


Figure 7-20. Reinforcement of Section A-A

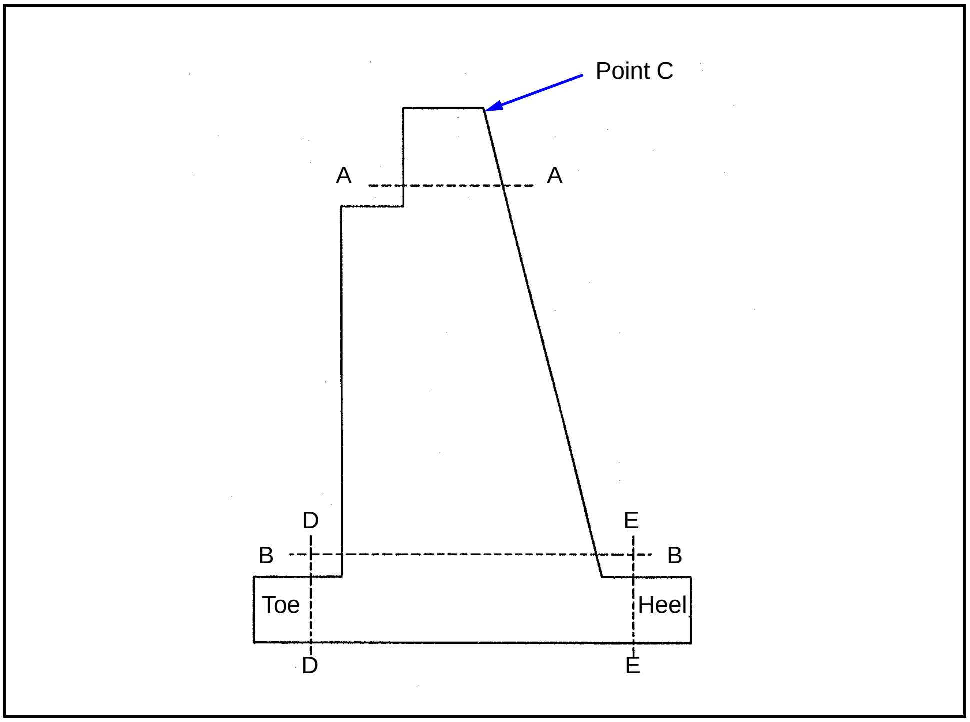


Figure 7-21. Concrete-Stress Checkpoints

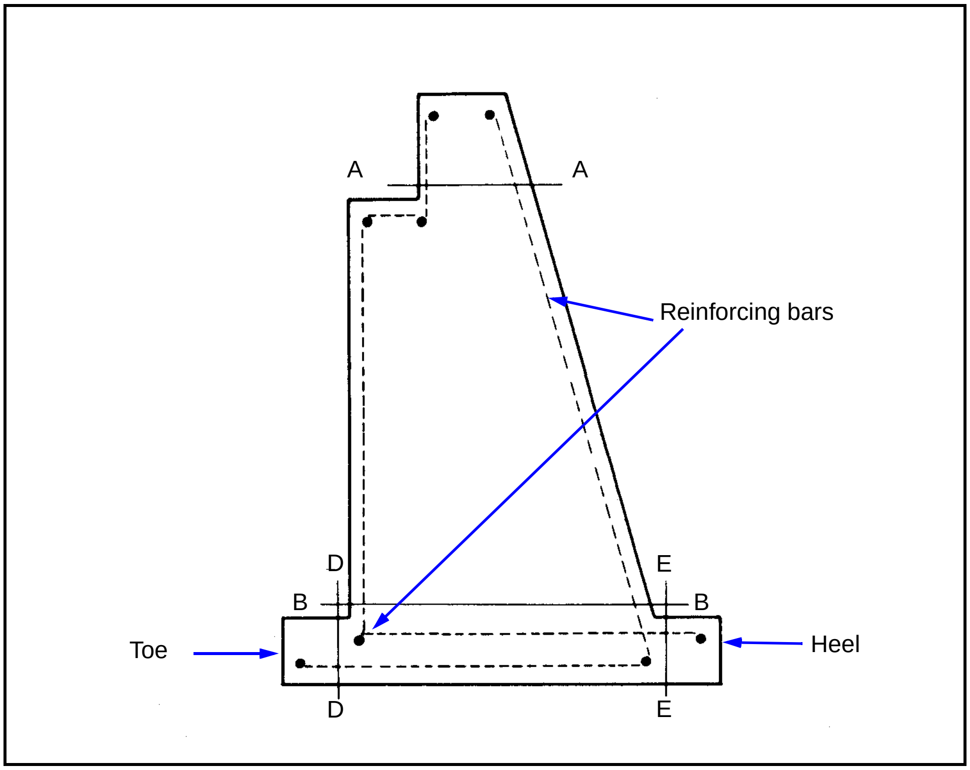


Figure 7-22. Reinforced Concrete Abutment With Critical Cross Sections

Loading Case Values

7-49. Using the preliminary dimensions outlined above, determine the vertical and horizontal abutment loads using the methods described in *paragraphs 7-2* and *7-8*. Determine the soil pressure at Points A and D *(Figure 7-23)* using the methods in *paragraphs 7-42* and *7-43*. The maximum pressure of eccentricity must be less than or equal to the soil bearing capacity, and the minimum pressure of eccentricity must be greater than zero for all loading cases.

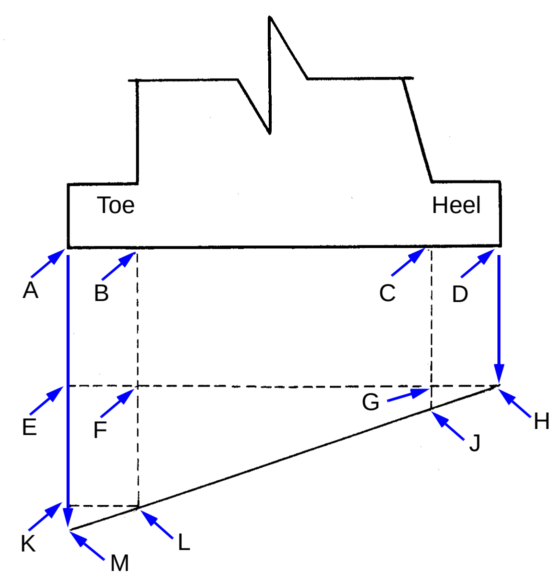


Figure 7-23. Soil-Pressure Points

Shear and Moment

7-50. Determine the shear force and moment as shown below. Each section is computed separately.

7-51. **Sections A-A and B-B.** Determine the shear force and the moment for Sections A-A and B-B *(Figure 7-22, page 7-25)* using the equation below. For Sections A-A and B-B, consider only that portion of the surcharge force plus the backfill force that acts at or above the respective section.

- **Shear force.**

$$V = H_s + H_b \tag{7-31}$$

where—

V = *shear force of Section A-A or B-B, in kpf*

H_s = *horizontal surcharge force above either Section A-A or B-B, in kpf (equation 7-9)*

H_b = *horizontal backfill force above either Section A-A or B-B, in kpf (equation 7-10)*

- **Moment.**

$$M = Ve \tag{7-32}$$

where—

M = *total moment of Section A-A or B-B, in kip-feet per foot (Figure 7-22)*
V = *total of the shear forces, in kpf (equation 7-31)*
e = *eccentric distance, in feet (equation 7-28)*

7-52. **Section D-D.** The shear force at Section D-D *(Figure 7-22*) is the total of the forces exerted at the centroid of the rectangle ABLK and the triangle KLM in *Figure 7-23*. Compute for shear force and moment as follows:

- **Shear force.**

$$V_1 = P_{mn}b_t + (P_{mx} - P_{mn})\left(\frac{b_a - b_t}{b_a}\right)(b_t) \text{ @ } \frac{b_t}{2} \tag{7-33}$$

and—

$$V_2 = (P_{mx} - P_{mn})\left(\frac{b_t}{b_a}\right)\left(\frac{b_t}{2}\right) \text{ @ } \frac{2b_t}{3} \tag{7-34}$$

where—

V_1 = *shear force exerted by the rectangle ABLK, in kpf*
P_{mn} = *minimum pressure on the soil, in ksf (equation 7-30)*
b_t = *toe width, in feet*
P_{mx} = *maximum pressure on the soil, in ksf (equation 7-29)*
b_a = *abutment width, in feet*
V_2 = *shear force exerted by the triangle KLM, in kpf*

- **Moment.**

$$M = V_1\left(\frac{b_t}{2}\right) + V_2\left(\frac{2b_t}{3}\right) \tag{7-35}$$

where—

M = *total moment of Section D-D, in kip-feet per foot*
V_1 = *shear force exerted by the rectangle ABLK, in kpf (equation 7-33)*
b_t = *toe width, in feet*
V_2 = *shear force exerted by the triangle KLM, in kpf (equation 7-34)*

7-53. **Section E-E.** The shear force at Section E-E *(Figure 7-22*), resulting in tension at the bottom of the slab, is the weight of the soil plus the surcharge above the heel and the weight of the heel minus the forces exerted at the centroid by the rectangle CDHG and the triangle GHJ. Compute for shear force and moment as follows:

- **Shear force.**

$$V_1 = P_{mn}b_h \tag{7-36}$$

and—

$$V_2 = (P_{mx} - P_{mn})\left(\frac{b_h}{b_a}\right)\left(\frac{b_h}{2}\right) \tag{7-37}$$

where—

V_1 = *shear force exerted by the rectangle CDHG, in kpf*
P_{mn} = *minimum pressure on the soil, in ksf (equation 7-30)*
b_h = *heel width, in feet*
V_2 = *shear force exerted by the triangle GHJ, in kpf*
P_{mx} = *maximum pressure on the soil, in ksf (equation 7-29)*
b_a = *abutment width, in feet*

- **Moment.**

$$M = V_1\left(\frac{b_h}{2}\right) + V_2\left(\frac{b_h}{3}\right) \tag{7-38}$$

where—

M = *total moment of Section E-E, in kip-feet per foot*
V_1 = *shear force exerted by the rectangle CDHG, in kpf (equation 7-36)*
b_h = *heel width, in feet*
V_2 = *shear force exerted by the triangle GHJ, in kpf (equation 7-37)*

Critical Sections

7-54. The design procedure for critical sections corresponds to the procedure used for flat slabs with reinforcement in one direction. Base the design on a 1-foot-wide section. At each critical section, check for moment, shear, and bond (between steel and concrete). Assume that there are 10 stringers and that the ultimate compressive stress is 3 ksi, the allowable concrete stress is 1.35 ksi, and the allowable steel stress is 24 ksi. Compute as follows:

- **Required section depth.**

$$d_{req} = 2.16\sqrt{M} \tag{7-39}$$

where—

d_{req} = *required section depth, in inches (round to the next higher whole inch)*
M = *design moment, in kip-feet per foot (equation 7-32, 7-35, or 7-38)*

- **Required steel area.** The total depth of Section E-E *(Figure 7-22, page 7-25)* is the required section depth plus 3 inches. This depth provides protection for the reinforcing steel. Once the steel area is known, choose the bar sizes from *Appendix D*.

$$A = \frac{M}{1.76d} \tag{7-40}$$

where—

A = *required steel area, in square inches*
M = *moment at any critical section of abutment (in 1-foot-wide sections), in kip-feet (equation 7-32, 7-35, or 7-38)*
d = *actual abutment depth, in inches*

- **Actual shear stress.** If the actual shear stress is critical, increase the required section depth to decrease the shear stress. Ensure that the actual shear stress does not exceed the allowable shear stress (0.09 ksi).

$$f_v = \frac{V}{10.5d_{req}} \tag{7-41}$$

where—

f_v = *actual shear stress, in ksi*
V = *total shear force at the critical section, in kpf (paragraph 7-51)*
d_{req} = *required section depth, in inches (equation 7-39)*

- **Bond stress.** Check the bond stress of the reinforcing bars against the allowable bond stress (0.03 ksi). Compute as follows:

$$f_o = \frac{V}{\Sigma_O(0.875d_{req})} \tag{7-42}$$

where—

f_o = *allowable bond stress, in ksi*
V = *total shear at the critical section, in kpf (paragraph 7-51)*
Σ_o = *sum of all the perimeters of the reinforcing bar, in inches (Table D-5, page D-5)*
d_{req} = *required section depth, in inches (equation 7-39)*

Sliding and Overturning

7-55. Use the methods described in *paragraphs 7-29* and *7-30* to check for sliding and overturning. If either sliding or overturning presents a problem, increase the heel width to increase the total weight of soil over the heel. Also use the measures described in *paragraph 7-31* to prevent sliding and overturning. If the safety factors for both sliding and overturning are much larger than required and the soil bearing capacity is more than adequate, decrease the size of the originally designed abutment to provide a more economical design.

Soil Shear Failure

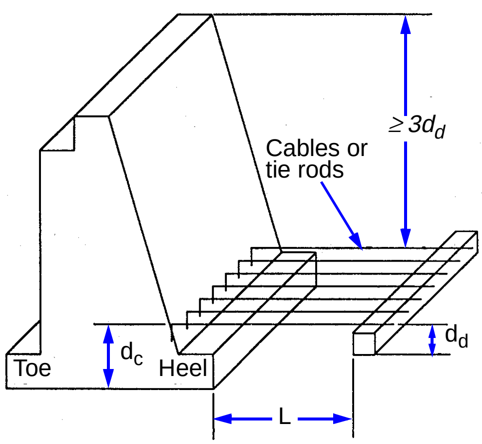
7-56. Apply the measures described in *paragraph 7-44* to reinforced concrete abutments. This will prevent soil shear failure.

RETAINING WALLS

7-57. Design retaining walls the same as abutments, but do not apply superstructure dead and live loads to the top of the retaining walls. Retaining walls may be concrete (mass or reinforced), timber, or steel sheet piles.

DEADMAN DESIGN

7-58. Use a deadman as a means of preventing sliding and overturning. Place the deadman outside the natural angle of repose of the soil. A setback equal to 150 percent of the abutment height is sufficient. Anchor the cables or steel tie rods by casting them into the body of the concrete abutment as shown in *Figure 7-24*. If the bridge is constructed of piles or posts, bolt the cables or tie rods to the abutment as shown in *Figure 7-25*. Position the cables or tie rods horizontally between the abutment and the deadman. The cover over the cables should be three times the deadman's depth dimension or the depth of the covered stem plus 1 foot, whichever is greater.



LEGEND:

d_c = distance from abutment bottom to cable penetration
d_d = deadman depth
L = deadman setback (150 percent of abutment height)

Figure 7-24. Deadman Connection, Cover, and Setback

Sliding

7-59. Use *equation 7-23* to check for sliding on a concrete abutment. If the safety factor is less than 1.5, design a deadman as follows:

- **Concrete or footing-type abutment.**
  - Compute the horizontal force acting on the abutment as follows:

$$\Sigma H_M = \frac{\Sigma P K_f L_a}{1.5} \qquad (7\text{-}43)$$

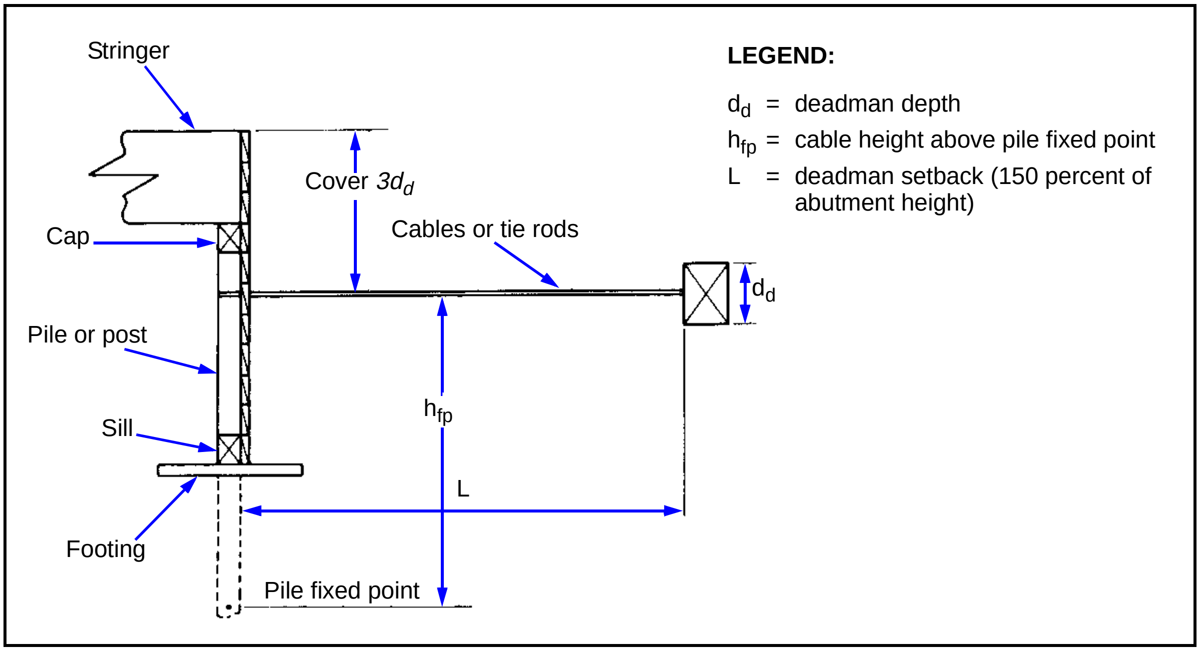


Figure 7-25. Deadman Installation on a Pile or Post Abutment

where—

ΣH_M = *maximum horizontal force acting on the abutment, in kips*
ΣP = *total of the vertical loads, in kpf (paragraph 7-2)*
K_f = *friction coefficient (Table H-2, page H-2)*
L_a = *abutment length, in feet (Figure 7-13, page 7-16)*

- Compute the horizontal force that the deadman must resist as follows:

$$H'_d = \Sigma H - \Sigma H_M \qquad (7\text{-}44)$$

where—

H'_d = *horizontal force that the deadman must resist, in kips*
ΣH = *total of the horizontal forces acting on the abutment, in kpf (paragraph 7-8)*
ΣH_M = *maximum horizontal forces acting on the abutment, in kips (equation 7-43)*

- Use a safety factor of 1.5 to determine the capacity of the deadman to resist horizontal forces. Compute as follows:

$$H_d = 1.5H'_d \qquad (7\text{-}45)$$

where—

H_d = *horizontal-force capacity of the deadman, with safety factor, in kips*

H'_d = *horizontal force that the deadman must resist, in kips*

- **Pile abutment.**
  - Compute the allowable shear capacity per pile as follows:

$$h_{av} = F_v A_v \tag{7-46}$$

where—

h_{av} = *allowable horizontal shear capacity of the pile, in kips*
F_v = *allowable shear stress, in ksi (Table C-1, pages C-3 through C-6)*
A_v = *effective shear area per pile, in square inches (Table C-4, page C-9)*

  - Compute the total horizontal force resisting shear as follows:

$$H_{av} = N h_{av} \tag{7-47}$$

where—

H_{av} = *total horizontal force resisting shear, in kips*
N = *number of piles in the bent*
h_{av} = *horizontal shear capacity of the pile, in kips (equation 7-46)*

  - Compute the horizontal force that the deadman must resist, in *equations 7-44* and *7-45,* substituting H_{av} for ΣH_M.

Overturning

7-60. Use *equation 7-24* to compute the safety factor for overturning. If it is less than two, design the abutment as follows:

- **Compute the concrete or footing-type abutment.**

$$\Sigma M_r = 2\Sigma M_o \tag{7-48}$$

where—

ΣM_r = *minimum resisting moment of the abutment, in kips*
ΣM_o = *total of the overturning moments of the abutment sections, in kips (Figure 7-12, page 7-12)*

- **Compute the pile abutment.**

$$M_r = \frac{N_p F_b S}{12} \tag{7-49}$$

where—

M_r = *minimum resisting moment per pile, in kip-feet*
N_p = *number of piles*
F_b = *allowable bending stress per pile, in ksi (Table C-1)*
S = *section modulus of the pile, in cubic inches (Table C-5, page C-10)*

7-61. Design the deadman to resist the horizontal shear force, including a safety factor of two. Design the deadman to resist the larger force required for sliding and overturning. Compute as follows:

- **Concrete.**

$$H_d = 2\left[\frac{L_a(\Sigma M_r - \Sigma M_o)}{h}\right] \tag{7-50}$$

where—

H_d = *concrete capacity of the deadman to resist horizontal shear, in kips*
L_a = *abutment length, in feet*
M_r = *minimum resisting moment of the abutment, in kip-feet (equation 7-48)*
M_o = *total of section moments, in kip-feet per foot (Figure 7-12)*
h = *distance from the cable to the bottom of the abutment, in feet (Figures 7-24 and 7-25, pages 7-30 and 7-31)*

- **Piles.**

$$H_d = 2\left[\frac{N(\Sigma M_r - \Sigma M_o)}{h}\right] \tag{7-51}$$

where—

H_d = *pile capacity of the deadman to resist horizontal shear, in kips*
N = *number of piles*
M_r = *minimum resisting moment per pile, in kip-feet (equation 7-49)*
M_o = *overturning moment, in kip-feet (Case III, paragraphs 7-2 through 7-17 and Figure 7-12)*
h = *distance from the cable to the bottom of the abutment, in feet (Figures 7-24 and 7-25)*

Deadman Specifications

7-62. **Length and Depth.** The deadman length normally equals the abutment length. Compute the required structural depth of the deadman as follows:

$$d_d = \frac{H_d}{L_d F_s} \tag{7-52}$$

where—

d_d = *required structural deadman depth, in feet*
H_d = *capacity of the deadman needed to resist horizontal forces, in kips (larger of equations 7-45 and 7-50 or 7-51)*
L_d = *deadman length, in feet*
F_s = *allowable soil bearing capacity, in ksf (Table H-1, page H-1)*

7-63. **Number of Cables.** Compute the number of cables using the equation below. If using steel rods instead of cables, use the same equation but substitute the rod strength *(rod area x 29 ksi)* for the allowable rod strength.

$$N = \frac{H_d}{T} \qquad (7\text{-}53)$$

where—

N = *number of cables*
H_d = *capacity of the deadman needed to resist horizontal forces, in kips (larger of equations 7-45 and 7-50 or 7-51)*
T = *allowable cable tensile strength, in kips (16 ksi x the square of the cable diameter)*

7-64. **Cable or Rod Spacing.** Compute the proper spacing and deadman width as follows:

- **Spacing.**

$$S_{cr} = \frac{b_R}{N} - 1 \qquad (7\text{-}54)$$

where—

S_{cr} = *cable or rod spacing, in feet*
b_R = *curb-to-curb roadway width, in feet (equation 7-14)*
N = *number of cables or rods*

- **Width.**

$$b_w = \frac{S_{cr}}{5} \qquad (7\text{-}55)$$

where—

b_w = *deadman width, in feet*
S_{cr} = *cable or rod spacing, in feet (equation 7-54)*

7-65. **Cable Protection.** After installation, coat the cables or rods with tar to protect them from rust. Treat the deadman for protection against decay.

INTERMEDIATE-SUPPORT DESIGN

VERTICAL PIER LOADS

7-66. Vertical pier loads include dead and live loads, pier weight, soil forces, and buoyancy forces. The dead load acts vertically through the centerline of the support.

Dead Load

7-67. Compute the dead load of the pier as follows:

$$P_{DL} = \left(\frac{W'_{DL1}L_1}{2}\right)\left(\frac{W'_{DL2}L_2}{2}\right) \qquad (7\text{-}56)$$

where—

P_{DL} = *dead load of the pier, in kips*
W'_{DL1} = *dead-load weight per foot of span 1, in kpf (equation 6-43)*
L_1 = *length of span 1, in feet*
W'_{DL2} = *dead-load weight per foot of span 2, in kpf (equation 6-43)*
L_2 = *length of span 2, in feet*

Live Load

7-68. Compute the live load of the pier as follows:

$$P_{LL} = V_{LL}N \tag{7-57}$$

where—

P_{LL} = *live load of pier, in kips*
V_{LL} = *maximum live-load shear, in kips (Use the larger value from Figure B-3 or B-4, pages B-17 and B-18. Use the combined span length for determining the shear value in the figures.)*
N = *number of lanes*

7-69. Note that the two span lengths resting on the pier are added, and the live-load shear is found for the combined span lengths. Impact is not included in this load. The live load acts vertically through the centerline of the pier.

Pier Weight

7-70. The weight of timber- or steel-framed piers is negligible. However, include the weight of a concrete pier in the substructure design. Determine the concrete-pier weight from known or assumed dimensions. *Figure 7-26, page 7-36,* shows a typical concrete pier (with dimensions). The forces of the stem and footing weights act vertically through the pier centerline.

7-71. **Stem Weight.** Compute the stem weight as follows:

$$W_s = L_s b_{as} h_{as} u \tag{7-58}$$

where—

W_s = *stem weight, in kips*
L_s = *stem length, in feet*
b_{as} = *stem width, in feet*
h_{as} = *stem height, in feet*
u = *unit weight of the stem material, in kips per cubic foot (concrete weighs 0.15 kips per cubic foot)*

7-72. **Footing Weight.** Compute the footing weight using *equation 7-58*. Substitute the footing dimensions for the stem dimensions.

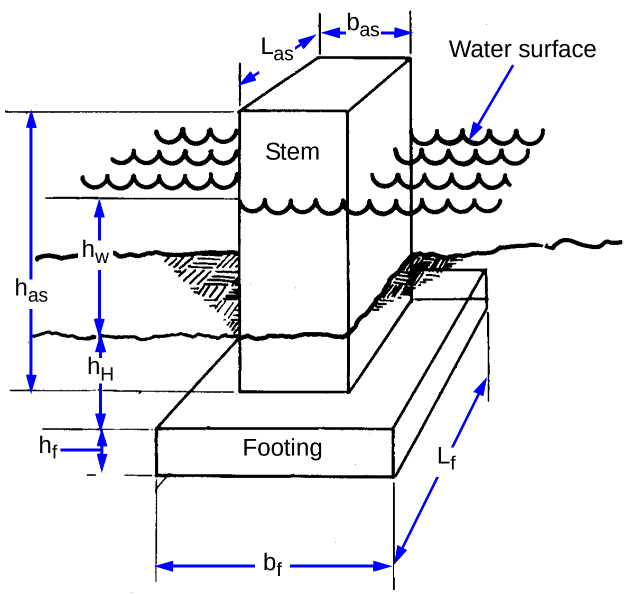


Figure 7-26. Typical Concrete Pier

Soil Forces

7-73. If there is a layer of soil over the footing, include the weight of the soil in the vertical pier load. Compute the soil weight over the pier (as shown in *Figure 7-26)* as follows:

$$W_k = (A_f - A)h_H u \tag{7-59}$$

where—

W_k = *soil weight, in kips*
A_f = *footing area, in square feet*
A = *stem area, in square feet*
h_H = *height of the soil over the footing, in feet (Figure 7-26)*
u = *unit weight of the soil, in kips per cubic foot (Table H-1, page H-1)*

7-74. If the stem is steel H-sections, the area of the steel is negligible. If the stem is timber, compute the soil weight as follows:

$$W_k = (A_f - A_t)h_H u \tag{7-60}$$

where—

W_k = *soil weight, in kips*
A_f = *footing area, in square feet*
A_t = *planned area of the timber over the footing, in square feet*
h_H = *height of the soil over the footing, in feet (Figure 7-26)*
u = *unit weight of the soil, in kips per cubic foot (Table H-1)*

Buoyancy Forces

7-75. If the pier is partially submerged *(Figure 7-26)*, the water creates buoyancy forces on the submerged parts of the pier. The buoyancy force on any part is simply the volume of that part below water multiplied by the unit weight of water. Buoyancy forces for steel or timber piers are negligible. Compute for the buoyancy force acting on the pier stem as follows:

$$F_{os} = [A(h_w + h_H)]u \tag{7-61}$$

where—

F_{os} = *buoyancy force acting on the stem, in kips*
A = *stem area, in square feet*
h_w = *water depth, in feet*
h_H = *height of the soil in contact with the stem, in feet (Figure 7-26)*
u = *unit weight of water, in kips per cubic foot (salt water is 0.064 and freshwater is 0.0624)*

7-76. Similarly, the buoyancy forces acting on the footing and soil are computed as follows:

$$F_{of} = L_f b_f h_f u \tag{7-62}$$

where—

F_{of} = *buoyancy force acting on the footing, in kips*
L_f = *footing length, in feet*
b_f = *footing width, in feet*
h_f = *submerged footing height, in feet*
u = *unit weight of water, in kips per cubic foot (salt water is 0.064 and freshwater is 0.0624)*

and—

$$F_{ok} = (A_f - A)du \tag{7-63}$$

where—

F_{ok} = *buoyancy force acting on the soil, in kips*
A_f = *footing area, in square feet*
A = *stem area, in square feet*
d = *submerged soil depth over footing, in feet (Figure 7-26)*
u = *unit weight of water, in kips per cubic foot (salt water is 0.064 and freshwater is 0.0624)*

Total Vertical Pier Load

7-77. The total vertical pier load is the sum of all vertical loads minus the buoyancy forces. For the pier shown in *Figure 7-26*, the total vertical load is as follows:

$$P = P_{DL} + P_{LL} + W_s + W_f + W_k - (F_{os} + F_{of} + F_{ok}) \tag{7-64}$$

where—

P = *total vertical pier load, in kips*
P_{DL} = *dead load of the pier, in kips (equation 7-56)*
P_{LL} = *live load of the pier, in kips (equation 7-57)*
W_s = *stem weight, in kips (equation 7-58)*
W_f = *footing weight, in kips (paragraph 7-72)*
W_k = *soil weight, in kips (equation 7-59 or 7-60)*
F_{os} = *buoyancy force acting on the stem, in kips (equation 7-61)*
F_{of} = *buoyancy force acting on the footing, in kips (equation 7-62)*
F_{ok} = *buoyancy force acting on the soil, in kips (equation 7-63)*

LONGITUDINAL LOADS

7-78. The only longitudinal load considered in pier design is the vehicular longitudinal force. *Paragraph 7-12* describes how to determine the number of vehicles per lane.

LATERAL LOADS

Wind Load

7-79. Determine the wind force as follows:

- **On vehicles.** At normal convoy spacing of 100 feet, the wind load on vehicles is negligible.
- **On the substructure.** The effects of wind on the substructure are taken into account when determining the wind load on the superstructure.
- **On the superstructure.** Short bridges are designed with no allowance for wind. For bridges with combined span lengths of 100 feet or more, compute the wind force acting at each pier using the equation below. The safety factor (1.5) accounts for wind acting on the substructure, curbs, and handrails and the fact that wind acts with reduced force on stringers behind the windward stringer. Wind force acts at the top of the windward stringer as shown in *Figure 7-27*.

$$F_w = 0.03(1.5d)\frac{L_1 + L_2}{2} \qquad (7\text{-}65)$$

where—

F_w = *wind force on pier, in kips*
d = *superstructure depth (from the tread to the bottom of the stringer), in feet*
L_1 = *length of span 1, in feet*
L_2 = *length of span 2, in feet*

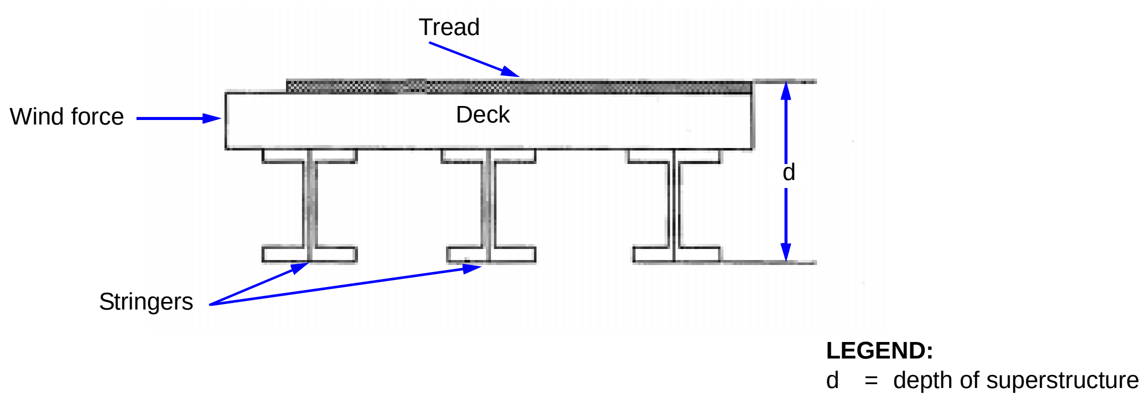


Figure 7-27. Wind Force on the Superstructure

Water Force on the Substructure

7-80. Water force acts on a pier at one-fourth the water height, measured downward from the water surface *(Figure 7-28, page 7-40)*. To determine the water force acting on a pier, first compute the area of the pier or posts upon which the water force acts.

- Compute the area of a concrete pier on which the water force acts as follows:

$$A_s = b_s d \tag{7-66}$$

where—

A_s = *stem area on which water force acts, in square feet*
b_s = *stem width, in feet*
d = *water depth above the lowest point of scour, in feet (Figure 7-28)*

- Compute the area of a pile or post on which the water force acts as follows:

$$A = 2N_r D_p d \tag{7-67}$$

where—

A = *pile area on which water force acts, in square feet*
N_r = *number of rows of piles or posts*
D_p = *pile diameter, in feet*
d = *water depth above the lowest point of scour, in feet (Figure 7-28)*

- Compute the water force acting on a pier as follows:

$$F_w = \frac{K_f v_y^2 A}{1,000} \tag{7-68}$$

where—

F_w = *water force acting on the pier, in kips*
K_f = *friction coefficient of water on the pier (Table 7-3)*
v_y = *water velocity, in fps*
A = *contact area (concrete or pile), in square feet (equation 7-67)*

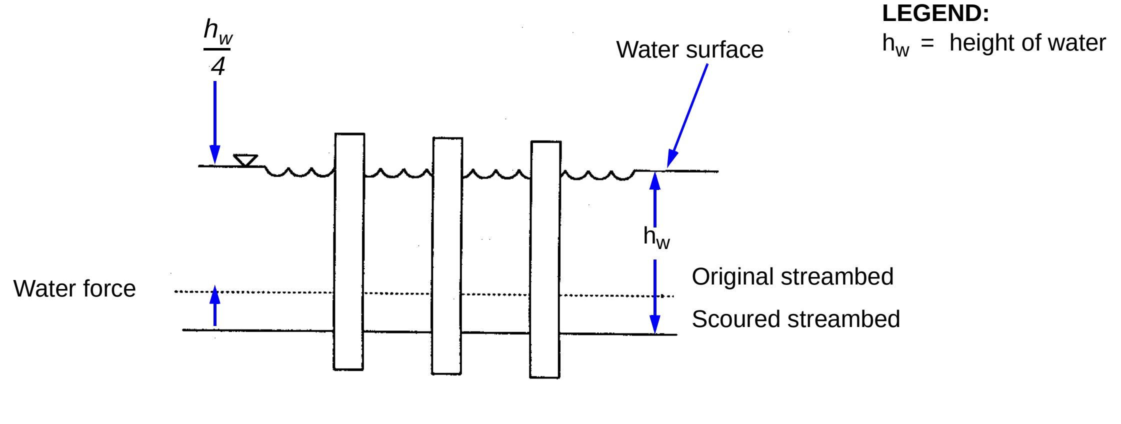


Figure 7-28. Water Force on the Substructure

Table 7-3. Pier Friction Coefficients

Pier Shape	Pier Friction Coefficients (K_f)
Square	1.4
Triangular	0.7
Round	0.7
Round piles	0.7
H-piles	1.4

Ice Forces

7-81. Consider the forces from ice as follows:

- **Crushing.** Ice crushing on concrete piers is negligible, since the compressive strength of concrete is much greater than the crushing force of ice. Timber piers, however, must be protected if the temperature falls below 0 degrees Fahrenheit (F). To protect the piers, attach steel angles to break up the ice *(Figure 7-29A).*

- **Thrust.** Ice thrust occurs at normal water level during winter months. Estimate this force using the equation below. For piers with dolphins *(Figure 7-29B),* reduce the ice thrust by 50 percent.

$$F_i = 0.4bt \quad (7\text{-}69)$$

where—

F_i = *ice thrust, in ksi*
b = *pier width, in inches*
t = *ice thickness, in inches*

- **Pileup.** Ice pileup occurs during the spring thaw when river ice breaks up and moves downstream in floes. These floes create large impact loads on anything in their path. In streams where ice pileup could occur, bridge piers should either be located out of the water or protected by dolphins.

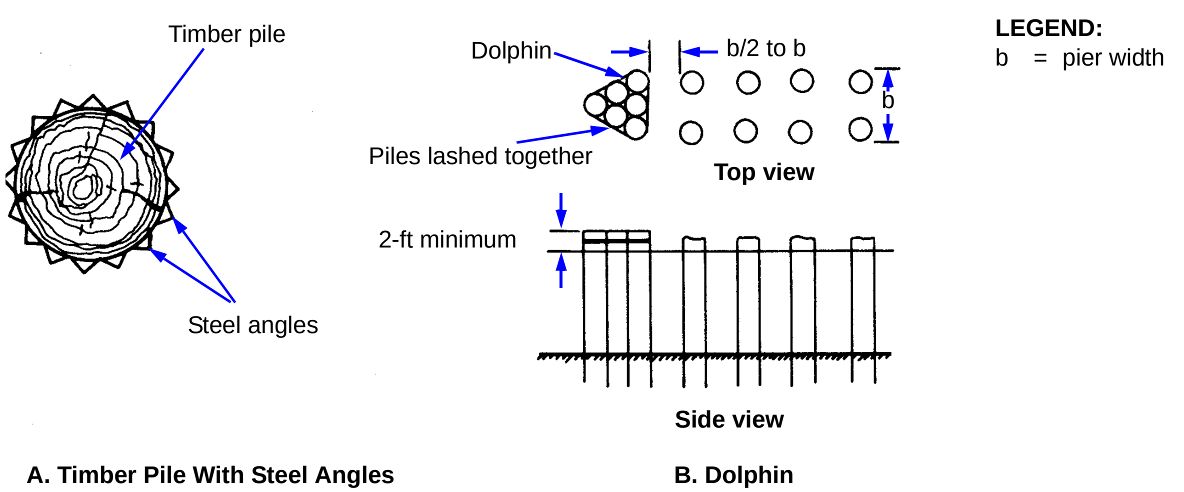


Figure 7-29. Pier Protection Devices

INTERMEDIATE-SUPPORT SELECTION

7-82. Intermediate supports may be constructed of timber, steel, concrete, or a combination of materials. They may be supported by footings or piles. *Figures 7-30* through *7-36, pages 7-42* through *7-45*, show various types of intermediate supports required for different conditions. *Table 7-4, page 7-45*, gives a general guide for selecting intermediate supports.

TIMBER-CRIB-PIER DESIGN

7-83. Timber-crib piers are assembled in log-cabin fashion *(Figure 7-30)*. For stability, the bottom of the crib is wider than the top. The base width is at least one-third the pier height. Drift bolts hold the timbers together at the corners. Fill the crib with rocks for ballast, if desired. For use in water, partially construct the crib on the shore, float it to position, and then sink it by filling it with rocks. Make the top of the pier level and solid to form a substantial support. Bridge stringers may rest directly on top of the crib. If more height is needed, construct a short timber-trestle bent or pier on top of the crib.

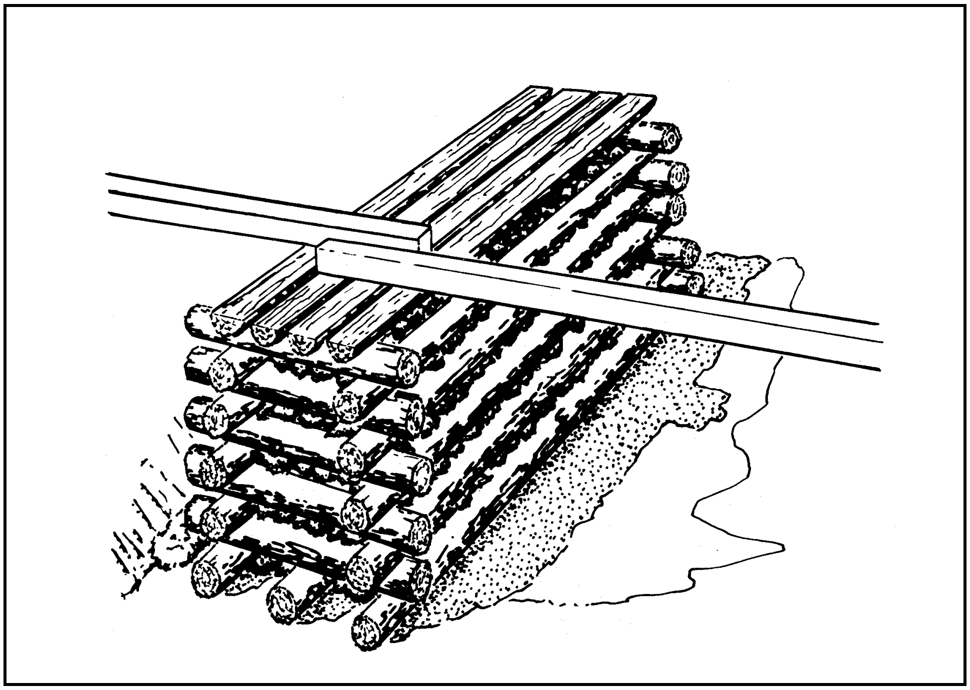


Figure 7-30. Timber-Crib Pier

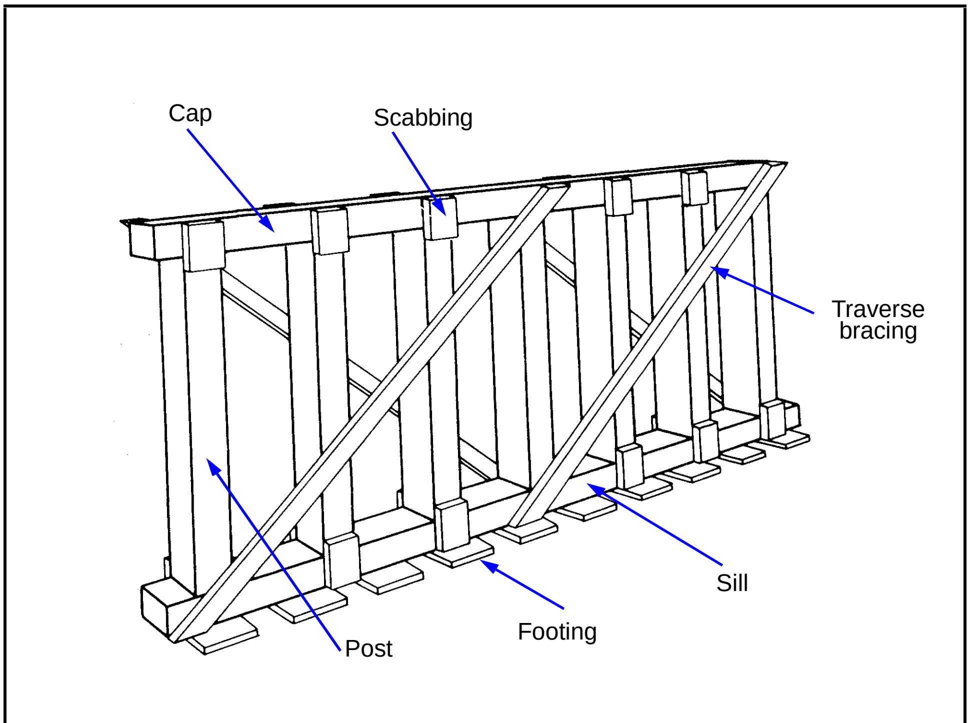


Figure 7-31. Timber-Trestle Bent

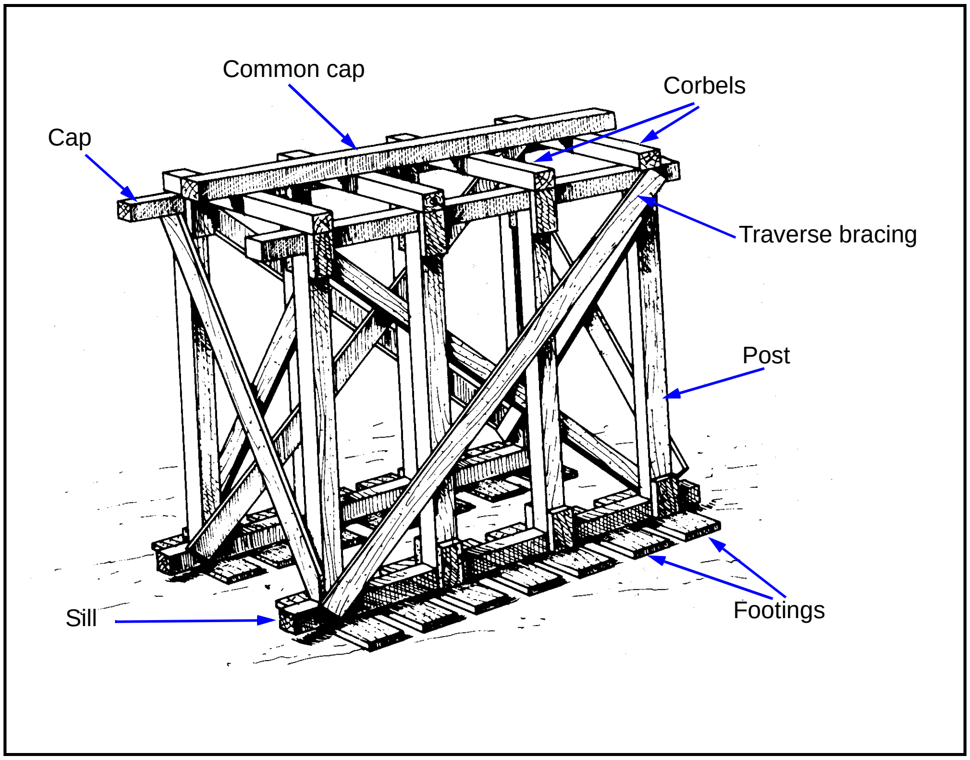


Figure 7-32. Timber-Trestle Pier

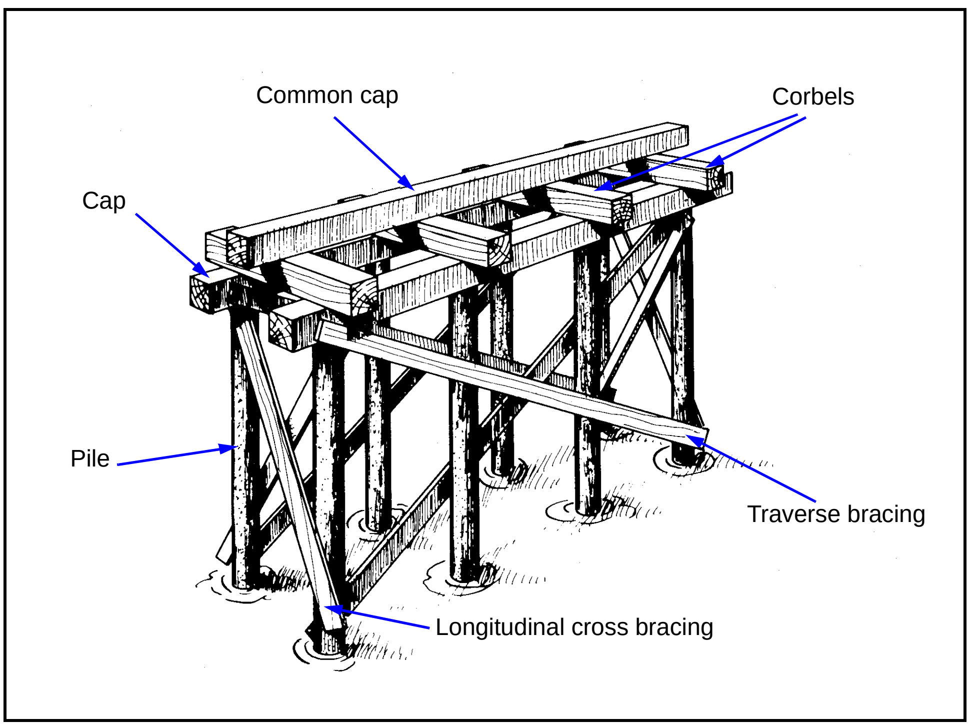


Figure 7-33. Pile Pier

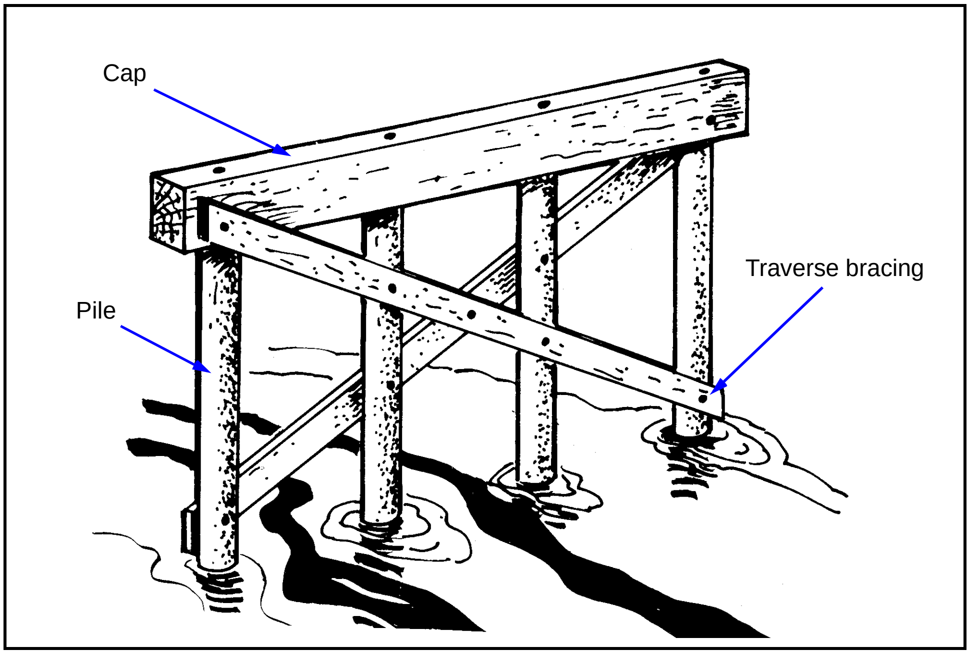


Figure 7-34. Pile Bent

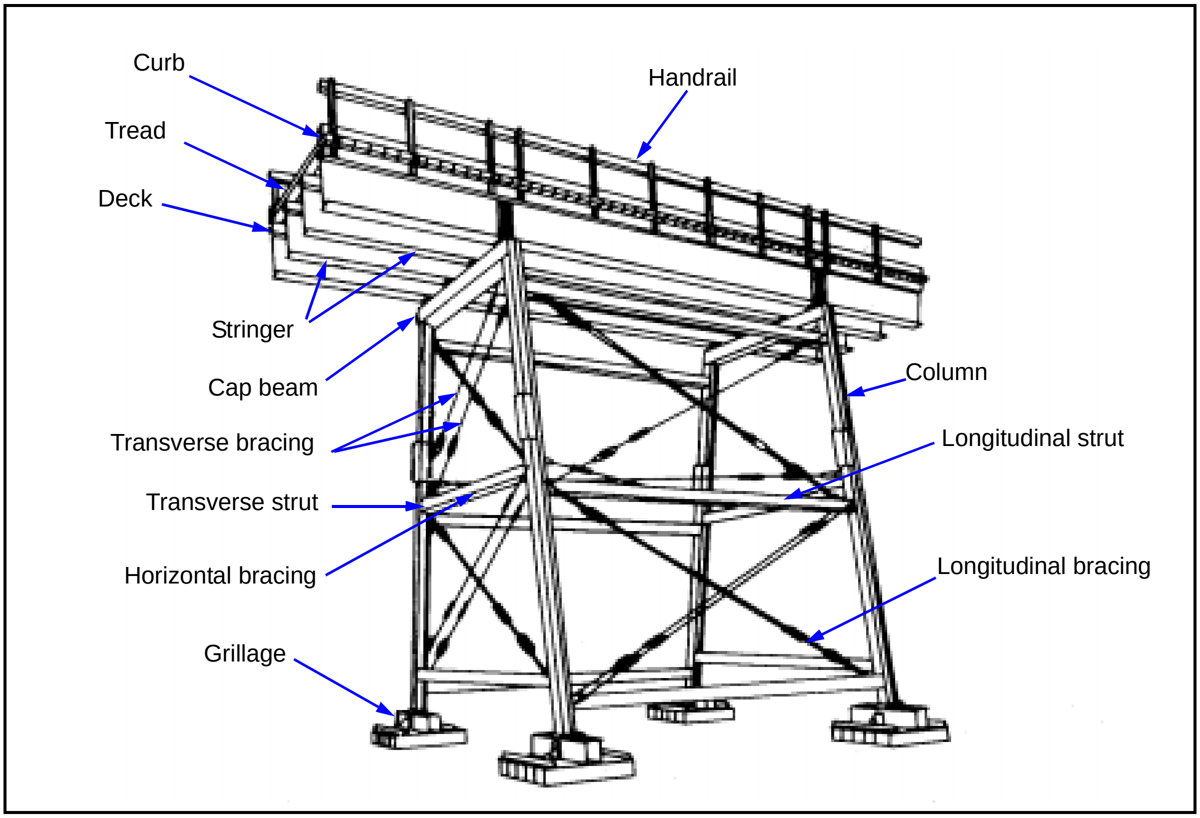


Figure 7-35. Steel-Framed Tower

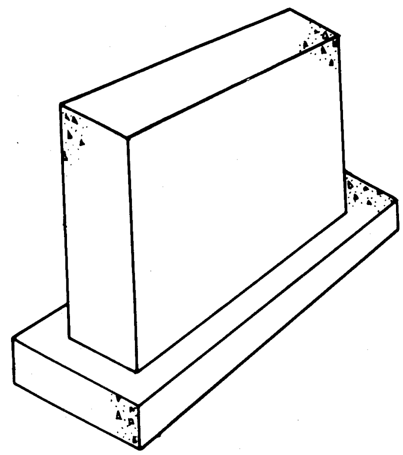


Figure 7-36. Concrete Pier

Table 7-4. Intermediate-Support Selection Guide

Type	Combined Span Length	Grade Height	Remarks
Timber-crib pier	To 50 feet	To 12 feet	Highway bridges only. Designed for vertical loads only. Steel or timber stingers.
Timber-trestle bent	To 30 feet	To 12 feet	Highway bridges only. Designed for vertical loads only. Steel or timber stringers.
Timber-trestle pier	To 60 feet	To 18 feet	Highway bridges only. Designed for vertical loads only. Steel or timber stringers.
Timber-pile bent	To 50 feet	Governed by unbraced length	Highway bridges only. Designed for vertical and lateral loads. Steel or timber stringers.
Timber-pile pier	To 200 feet	Governed by unbraced length	Highway and RR bridges. Designed for vertical and lateral loads. Steel or timber stringers.
Steel-pile bent	To 70 feet	Governed by unbraced length	Highway bridges only. Designed for vertical and lateral loads. Steel or timber stringers.
Steel-pile pier	Any length	Governed by unbraced length	Highway and RR bridges. Designed for vertical and lateral loads. Steel or timber stringers.
Framed-timber tower	Any length	To 60 feet	Highway and RR bridges. Designed for vertical and lateral loads. Steel or timber stringers.
Framed-steel tower	Any length	To 80 feet	Highway and RR bridges. Designed for vertical and lateral loads. Steel or timber stringers.
Concrete pier	Any length	To 25 feet	Highway and RR bridges. Designed for vertical and lateral loads. Steel or timber stringers.

TIMBER-TRESTLE-BENT AND -PIER DESIGN

7-84. Timber-trestle bents and piers *(Figures 7-31* and *7-32, pages 7-42* and *7-43)* are not suitable for use in soft soil or swift or deep watercourses. Construct them in dry, shallow gaps with firm soil. When longitudinal spacing between bents exceeds 25 feet, bracing becomes cumbersome. To provide for greater support and longitudinal stability for longer spans, use timber-trestle piers. The design of timber-trestle piers is the same as the bent design except for the cap and corbel system *(paragraph 7-120)*. Use the procedures discussed below to design a timber-trestle bent.

Loads

7-85. Use the procedures in *paragraph 7-66* to determine dead and live loads. Compute the total design load as follows:

$$P = P_{DL} + \frac{P_{LL}N_s}{4} \qquad (7\text{-}70)$$

where—

P = *total design load of the bent, in kips*
P_{DL} = *dead load of the pier, in kips (equation 7-56)*
P_{LL} = *live load of the pier, in kips (equation 7-57)*
N_s = *number of stringers in the superstructure*

Size and Number of Members

7-86. **Posts.** The absolute minimum post size is 6 x 6 inches, and the minimum number of posts per bent is four.

- Compute the allowable bearing capacity per post as follows:

$$P_B = F_cA \qquad (7\text{-}71)$$

where—

P_B = *allowable bearing capacity of the post, in kips*
F_c = *allowable compression (parallel to the grain of the post material), in ksi (Table C-1, pages C-3 through C-6)*
A = *cross-sectional area of the post, in square inches (Table C-4, page C-9)*

- Limit the length of the post to ensure that it will not fail by buckling. Check as follows:

$$L \le 30b \qquad (7\text{-}72)$$

where—

L = *post length, in feet*
b = *post width (rectangular post) or 90 percent of the post diameter, in feet*

- Recompute the number of posts required as follows:

$$N_{pr} = \frac{P}{P_B} \tag{7-73}$$

and—

$$N_p = \frac{N_{pr}}{N_r} \tag{7-74}$$

where—

N_{pr} = *required number of posts in the bent*

P = *total design load on the bent, in kips (equation 7-70)*

P_B = *allowable bearing capacity per post, in kips (equation 7-71)*

N_p = *actual number of piles per row (minimum of four) (round up to the nearest whole number)*

N_r = *number of rows*

7-87. **Caps and Sills.** The absolute minimum size for caps and sills is 6 x 8 inches. The larger dimension is usually vertical. The cap and sill must also be at least as wide as the posts *(Figure 7-37, page 7-48)*. Determine the bearing failure, the post spacing, and the depth as follows:

- **Bearing failure.** Check the cap and sill for bearing failure. If bearing failure is a problem, increase the number of posts.

$$\frac{P}{N_{pr}A} < F_B \tag{7-75}$$

where—

P = *total cap or sill load, in kips*

N_{pr} = *required number of posts in the bent (equation 7-73)*

A = *cross-sectional area of post or pile, in square inches (Table C-4)*

F_B = *allowable bearing perpendicular to the grain of post or pile material, in ksi (Table C-1)*

- **Post spacing.** Compute post spacing the same as stringer spacing.

$$S_p = \frac{12b_R}{N_p - 1} \tag{7-76}$$

where—

S_p = *post or pile spacing, in inches*

b_R = *curb-to-curb roadway width, in feet*

N_p = *actual number of posts or piles in the bent (equation 7-74)*

- **Depth.**

$$d_c > \frac{S_p}{5} \tag{7-77}$$

where—

d_c = *cap or sill depth, in inches*

S_p = *post or pile spacing, in inches (equation 7-76)*

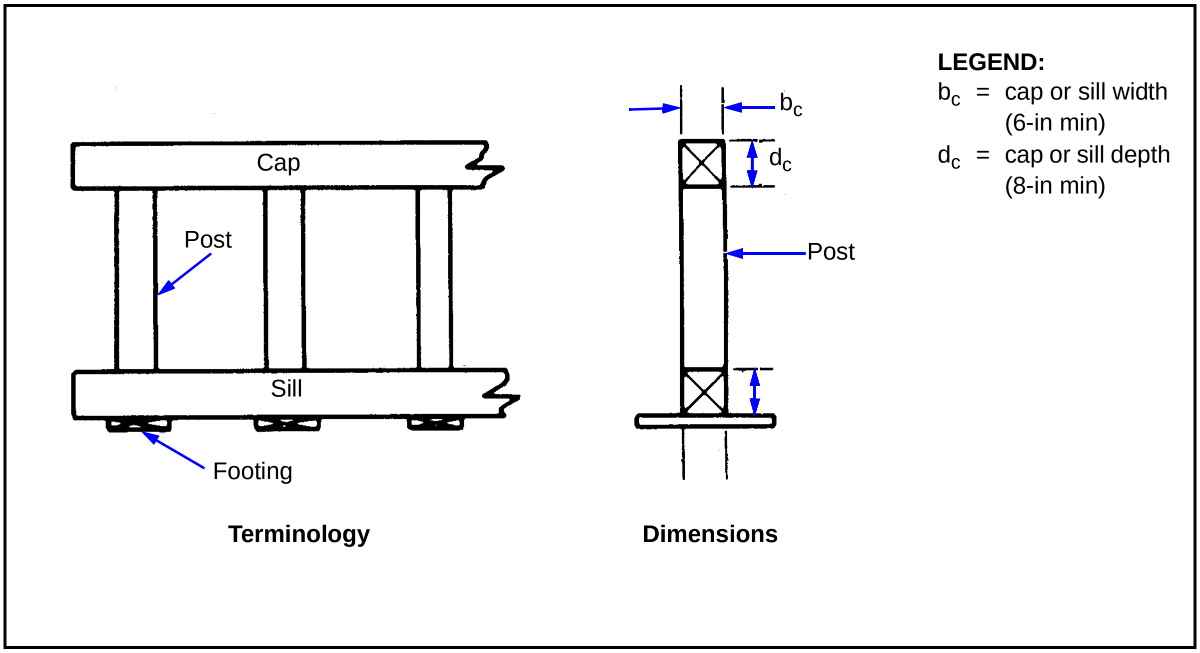


Figure 7-37. Limiting Dimensions for a Timber-Trestle Bent

7-88. **Footings.** The number of footings must equal or exceed the number of posts. A footing design for bents is identical to a footing design for abutments *(paragraph 7-22)*.

7-89. **Bracing.** Provide adequate longitudinal bracing between bents to ensure longitudinal stability of the bridge. The minimum bracing size is 2 x 12 inches.

PILES

7-90. A pile is a slender structural member that is forced into the ground to support vertical, horizontal, or inclined loads. Since one pile may not have the capacity to carry a certain load, several piles may have to be grouped together for the pile foundation. Use piles when placing a foundation on soft soil, in deep water, or in swift watercourses that are likely to scour. The factors discussed below will affect pile design.

Scour

7-91. Scour is the gradual removal of earth surrounding a pier or abutment by water action *(Figure 7-38)*. If a bridge has long spans, it needs intermediate piers, which obstruct the water flow and increase stream and scouring action. (When the stream velocity is great, or as much as 2 fps, the bottom may be disturbed.) For example, when a pile is inserted into a flowing watercourse, the turbulence and eddy currents created by the pier cause scour. If the axis of a pier is not parallel to the direction of flow, excessive scour will result. Minimize scour by ensuring proper alignment of piers in watercourses.

7-92. **Local Scour.** The scour depth varies with many factors. As a rule, estimate local scour depth as follows:

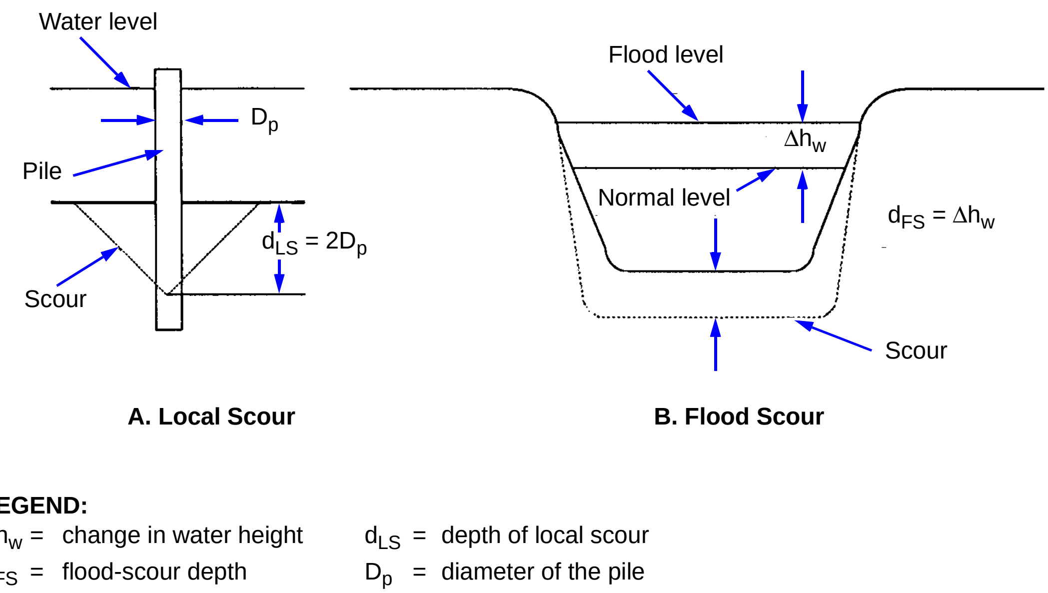


Figure 7-38. Types of Scour

$$d_{LS} = 2D_p \tag{7-78}$$

where—

d_{LS} = *local-scour depth, in feet*
D_p = *pile diameter, in feet*

7-93. **Flood Scour.** Consider flood scour in the substructure design because it is the cause of many bridge failures. If possible, determine the depth of flood scour at the site *(Figure 7-38B)*. If it cannot be determined, estimate the depth of flood scour as follows:

$$d_{FS} = \Delta h_w \tag{7-79}$$

where—

d_{FS} = *flood-scour depth, in feet*
Δh_w = *change in the water-surface elevation from normal to flood stages, in feet (Figure 7-38B)*

7-94. **Total Scour.** Total scour is the combination of local and flood scour. When the water surface drops after a flood, sediment is redeposited in the scoured area and the bottom returns to its original level. Compute the total scour as follows:

$$d_{TS} = d_{LS} + d_{FS} \tag{7-80}$$

where—

d_{TS} = *total-scour depth, in feet*

d_{LS} = *local-scour depth, in inches (equation 7-78)*

d_{FS} = *flood-scour depth, in inches (equation 7-79)*

7-95. **Scour Prevention.** Protect foundations from scour by—

- Locating bents or piers parallel to the direction of water flow.
- Placing sandbags around bents, piers, or abutments on the upstream sides.
- Placing riprap around bents, piers, or abutments.
- Driving a row of closely spaced pile fenders (dolphins) perpendicular to the water flow on the upstream side of the bent, pier, or abutment. Recognize that fenders may compound the problem of scour because they further restrict water flow. Fenders are most effective when their widths are small relative to the watercourses width.

Pile-Support Types

7-96. Pile-support types include end bearing, friction, and batter. Each is discussed below.

7-97. **End-Bearing Piles.** End-bearing piles *(Figure 7-39A)* are firmly seated on rock or hard strata. The entire support of the piles is provided by the hard strata so that the load carried is limited by the strength and unbraced length of the pile material.

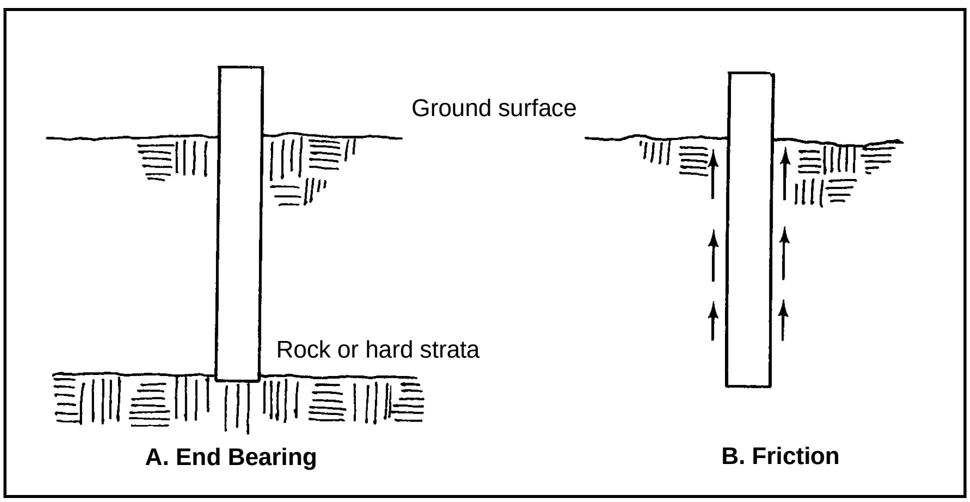


Figure 7-39. Types of Pile Support

7-98. **Friction Piles.** Friction piles *(Figure 7-39B)* derive their entire support from friction between the piles and the surrounding soil. The load a friction pile will carry depends on the properties of the soil and the strength of the pile material.

7-99. **Batter Piles.** Batter piles are driven into the ground at an angle *(Figure 7-40)*. The maximum slope at which a pile may be driven is 1:1 (horizontal to vertical), due to the limitations of the driving equipment. The normal slope for batter piles is 1:12. If the slope is within 1:12, the vertical load-carrying capacity will not have to be reduced. Using batter piles complicates the design of a pile foundation; therefore, design pile foundations for vertical piles. Then install batter piles as the outside piles for additional safety and stability.

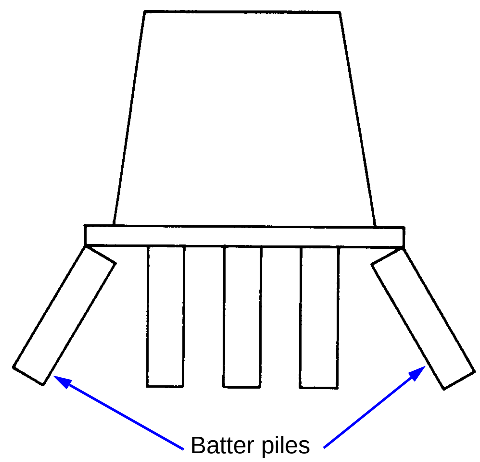


Figure 7-40. Batter Piles

PILE FOUNDATIONS

7-100. Pile foundations may be constructed of timber, steel, or concrete. Since concrete piles require special handling and equipment, they are not used in military construction. Design principles for timber and steel piles are the same except for the exceptions discussed below.

Allowable Load

7-101. For an end-bearing pile, the maximum allowable load is the smaller value of the allowable load or the buckling load of the pile. For a friction pile, the maximum allowable load is the smallest value of the allowable load, the buckling load, the soil friction capacity, or the pile-driving capacity. Check the bearing and buckling loads in both end-bearing and friction piles. Also, check the soil and pile-driving capacities in friction piles.

7-102. **Bearing Capacity.** The first step in designing a pile foundation is to determine the maximum allowable load that a single pile will carry. Compute the allowable bearing capacity of a pile as follows:

$$P_B = F_c A \tag{7-81}$$

where—

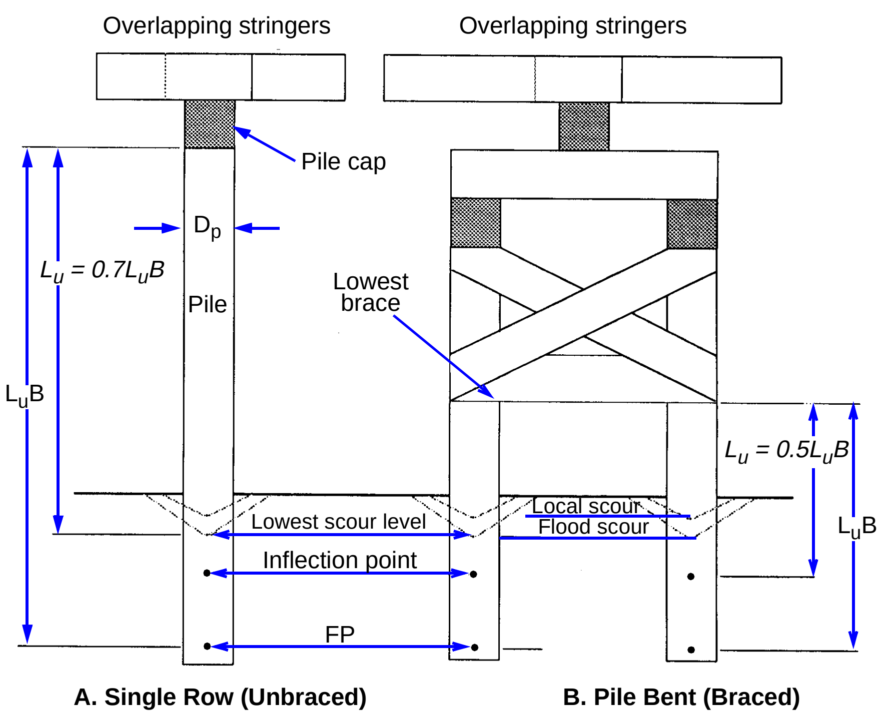
P_B = *allowable bearing capacity per pile, in kips*

F_c = *allowable compression (parallel to the grain of the pile material), in ksi (Table C-1, pages C-3 through C-6)*

A = *cross-sectional area of the pile, in square inches (Table C-5, page C-10)*

7-103. **Buckling Load.** The allowable buckling load for a single pile depends on the unbraced length of the pile, the size of pile, and the type of soil.

7-104. The fixed point (FP) is the point below which the pile is assumed to be completely rigid, so that any bending or buckling in the pile will take place above the FP *(Figure 7-41)*. The FP distance varies with the soil type (5 feet for sand and 10 feet for clay). Interpolate the FP for intermediate soils. Measure the FP distance from the point of lowest scour to ensure that the design accounts for the worst possible conditions.



LEGEND:

L_u = unbraced length

L_uB = length from the lowest brace to the FP

D_p = diameter of the pile

Figure 7-41. Maximum Unbraced Length

7-105. For a single row of piles unbraced in the longitudinal direction, the unbraced length is 70 percent of the distance from the FP to the top of the pile *(Figure 7-41A)*. For a single row of piles with adequate longitudinal bracing, the unbraced length is one-half the distance from the FP to the lowest bracing. For piles arranged in two or more rows with adequate bracing between, the unbraced length is one-half the distance from the FP to the lowest bracing *(Figure 7-41B)*.

- **Timber piles.** If the unbraced length divided by the pile diameter is less than or equal to 27, buckling is not a problem and no further checks are necessary. If the length divided by the pile diameter is greater than 27, find the allowable buckling load as follows:

$$P_{BU} = A\left[\frac{0.225E}{\left(\frac{L_u}{D}\right)^2}\right] \tag{7-82}$$

where—

P_{BU} = *buckling load, in kips*
A = *cross-sectional area of the pile, in square inches (Table C-5)*
E = *modulus of elasticity, in ksi (Table C-1)*
L_u = *unbraced length, in feet*
D = *pile diameter, in feet*

- **Steel piles (A36).** Compute the allowable buckling load as follows:

$$P_{BU} = A\left[(24.7 - 0.00077)\left(\frac{KL_u}{r}\right)^2\right] \tag{7-83}$$

where—

P_{BU} = *allowable buckling load, in kips*
A = *cross-sectional area of the pile, in square inches (Table C-5)*
K = *effective length factor (Table D-9, page D-9)*
L_u = *unbraced length, in inches*
r = *the least radius of gyration, in inches (Table D-6, page D-6)*

7-106. **Soil Capacity.** For friction piles, use load tests or static or dynamic formulas to find the allowable capacity of the soil surrounding the piles. Load tests are time-consuming and are justified only on large, permanent bridges. If the soil type is known, the soil capacity can be estimated using a static formula. To use this static formula, first find the area of the pile in contact with the soil.

- Compute the area for timber as follows:

$$A = \pi D L_g \tag{7-84}$$

where—

A = *area of the timber pile in contact with the soil, in square feet*
D = *pile diameter, in feet*
L_g = *length of pile in the soil, in feet*

- Compute the area for steel as follows:

$$A = 2(b_p + d_p)L_g \tag{7-85}$$

where—

A = *area of the steel pile in contact with the soil, in square feet*
b_p = *pile width, in feet*
d_p = *depth of pile section, in feet*
L_g = *length of pile in the soil, in feet*

- Compute the pile capacity (once the area of pile in contact with the soil is known) as follows:

$$P_f = f_s A \tag{7-86}$$

where—

P_f = *pile capacity (based on friction between pile and soil), in kips*
f_s = *allowable friction, in ksf (Table H-3, page H-2)*
A = *area of the pile in contact with the soil, in square feet (equation 7-84)*

7-107. If the soil conditions are not known, estimate the allowable soil capacity by driving a test pile and applying a dynamic formula to the results. Also use the dynamic formulas to check capacities estimated by the static formula. Dynamic formulas are only approximations; therefore, use them only if load tests are unavailable.

7-108. The basic assumption behind dynamic formulas is that driving resistance equals the static resistance of the pile to loads after driving is completed. However, this assumption is not always correct, and the more impervious the soil, the greater the discrepancy between the actual and computed loads. As a pile is driven, it squeezes water from the soil. Until the water drains from the surface of the pile, the full friction between the pile and the surrounding soil cannot develop. To lessen the discrepancies, let the pile rest for at least 24 hours. Redrive for at least 10 blows with a drop hammer or 30 blows with a pneumatic or diesel hammer. Use the penetration per blow after the pile has rested to estimate the allowable pile capacity.

7-109. **Driving Capacity.** Compute the driving capacity as follows:

- **Timber piles driven by a drop hammer.**

$$P_{TP} = \frac{2wh}{P_p + 1} \tag{7-87}$$

where—

P_{TP} = *driving capacity based on test pile, in kips*
w = *drop hammer or ram weight, in kips*
h = *average fall of the drop hammer, in feet*
P_p = *average pile penetration (last 6 blows of a drop hammer or last 20 blows of a powered drop hammer), in inches*

- **Steel piles driven by a drop hammer.**

$$P_{TP} = \frac{3wh}{P_p + 1} \tag{7-88}$$

where—

P_{TP} = *driving capacity based on test pile, in kips*
w = *drop hammer or ram weight, in kips*
h = *average fall of the drop hammer, in feet*

P_p = *average pile penetration (last 6 blows of a drop hammer or last 20 blows of a powered drop hammer), in inches*

- **Timber piles driven by a single-acting steam, pneumatic, or open-end diesel hammer.**

$$P_{TP} = \frac{2wh}{P_p + 0.1} \tag{7-89}$$

where—

P_{TP} = *driving capacity based on test pile, in kips*

w = *drop hammer or ram weight, in kips*

h = *average fall of the drop hammer, in feet*

P_p = *average pile penetration (last 6 blows of a drop hammer or last 20 blows of a powered drop hammer), in inches*

- **Steel piles driven by a single-acting steam, pneumatic, or open-end diesel hammer.**

$$P_{TP} = \frac{3wh}{P_p + 0.1} \tag{7-90}$$

where—

P_{TP} = *driving capacity based on test pile, in kips*

w = *drop hammer or ram weight, in kips*

h = *average fall of the drop hammer, in feet*

P_p = *average pile penetration (last 6 blows of a drop hammer or last 20 blows of a powered drop hammer), in inches*

- **Timber piles driven by a double-acting steam, pneumatic, or closed-end diesel hammer.**

$$P_{TP} = \frac{2E_H}{P_P + 0.1} \tag{7-91}$$

where—

P_{TP} = *driving capacity based on test pile, in kips*

E_H = *impact energy per blow of the hammer, in foot-pounds (Table 7-5, page 7-56)*

P_p = *average pile penetration (last 6 blows of a drop hammer or last 20 blows of a powered drop hammer), in inches*

- **Steel piles driven by a double-acting steam, pneumatic, or closed-end diesel hammer.**

$$P_{TP} = \frac{3E_H}{P_P + 0.1} \tag{7-92}$$

where—

P_{TP} = *driving capacity based on test pile, in kips*

E_H = *impact energy per blow of the drop hammer, in foot-pounds (Table 7-5)*

P_p = *average pile penetration (last 6 blows of a drop hammer or last 20 blows of a powered drop hammer), in inches*

Table 7-5. Impact Energy of Drop Hammers

Type	Strokes per Minute	Energy Foot-Pounds per Blow (E_H)
5,000-pound drop hammer	225	4,150
	195	3,720
	170	3,280
7,000-pound drop hammer	140	8,100
	130	6,800
	120	5,940

Overturning

7-110. Drive piles at least 8 feet into sand or 20 feet into clay to prevent overturning due to lateral loads. For intermediate soils, interpolate the distance.

Pile Groups

7-111. The spacing between rows of piles must equal or exceed three times the pile diameter. The spacing between rows must equal or exceed the spacing between piles. Each row must have at least four piles.

7-112. **Number of Piles Required.** Compute the number of piles as follows:

$$N_{pr} = \frac{P}{P_B} \qquad (7\text{-}93)$$

where—

N_{pr} = *required number of piles to support the vertical load*

P = *total design load on the bent, in kips (equation 7-13 or 7-70)*

P_B = *allowable bearing capacity per post, in kips. For end-bearing piles, use the smaller of the bearing and buckling loads. For friction piles, use the smallest of the bearing load, the buckling load, the soil capacity, or the driving capacity.*

7-113. **Group Action.** When several friction piles are driven close together, the interaction of pressure bulbs reduces the efficiency of each pile *(Figure 7-42)*. Use the Converse-Labarre method shown in *Figure 7-43, page 7-58,* to compute the effective number of piles in the group and to—

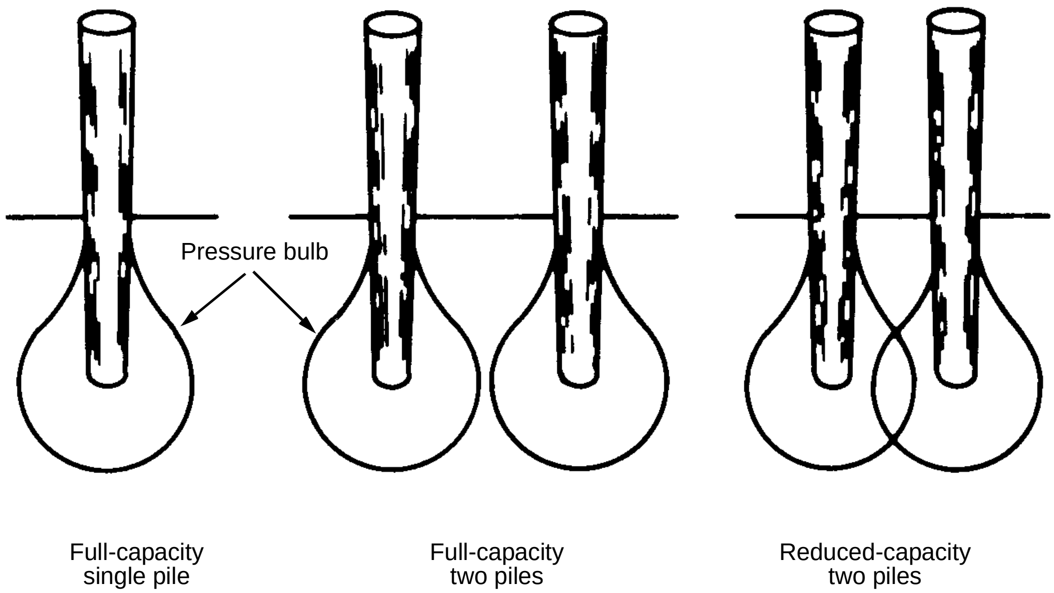
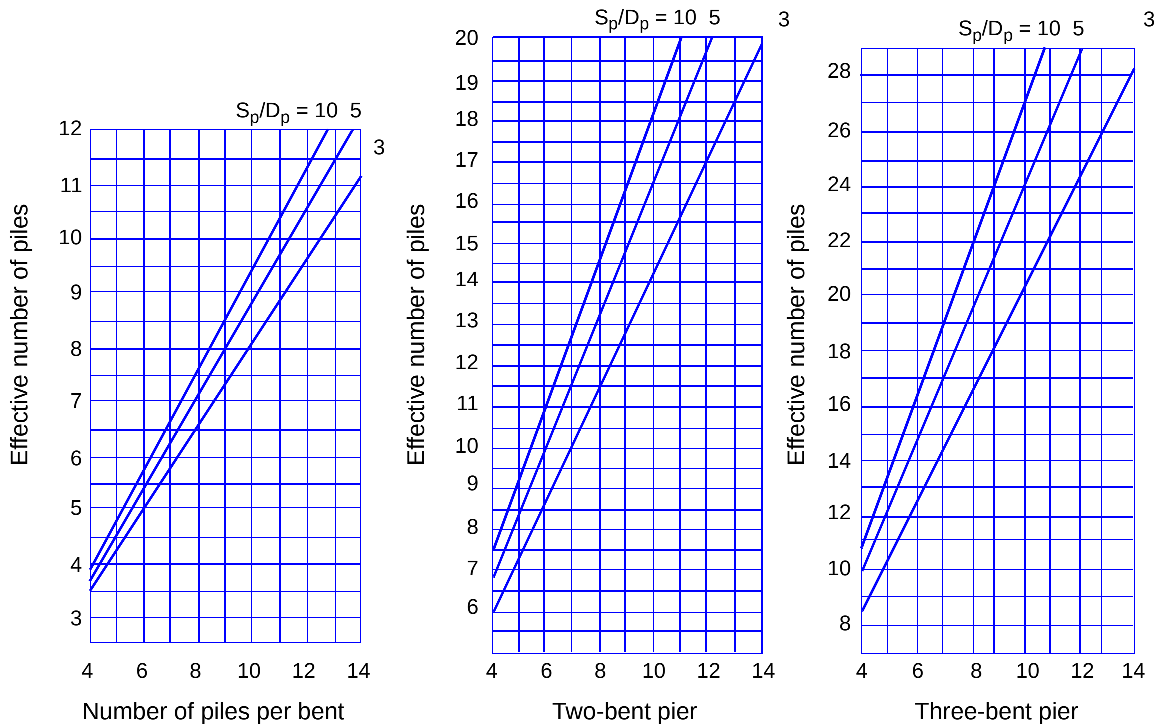


Figure 7-42. Effect of Grouping on Pile Capacity

- Estimate the number of bents. If each adjacent span exceeds 25 feet, a pile pier is needed. If the required number of piles is large, spacing considerations may require a three-bent pier.
- Use the number of bents to determine the theoretical spacing-to-diameter ratio (shown as S_p/D_p). Divide the number of piles required by the number of bents; then determine the theoretical S_p/D_p for the number of piles required per bent.
- Find the required number of piles along the *effective-number-of-piles* axis.
- Project a horizontal line to the right, intersecting the S_p/D_p.
- Interpolate the value of this ratio, with a minimum allowable value of three. Project a vertical line from the intersection of the *effective number of piles* and S_p/D_p to the bottom of the chart. Round the value for the number of piles up to the next higher whole number to get the actual number of piles per bent.
- Recheck the actual S_p/D_p with the rounded number of piles to ensure that it does not fall below three. Also, find the actual number of effective piles by reading the chart in reverse. This value must be greater than or equal to the required number of piles. The end-bearing piles will carry the full allowable pile load without any reduction in efficiency due to group action.



LEGEND:

S_p = center-to-center pile spacing

D_p = pile diameter

Figure 7-43. Converse-Labarre Method

7-114. **Combined Loading.** Wind, water, and ice are lateral forces that create combined loading conditions. Check the pile supports to ensure that the additional loads created by these forces will not cause failure. For example, compute the maximum actual load on an outside pile in a group as follows *(Figure 7-44, Pile A)*:

$$P_m = \frac{P}{N_p} + \frac{6\Sigma M}{S_p N_p (N_{pr} + 1)} \tag{7-94}$$

where—

P_m = *maximum actual load on the outside pile, in kips*

P = *total of the vertical loads on the pile, in kips (equation 7-70)*

N_p = *total number of piles in the group*

ΣM = *total moment of all lateral forces at the FP for both normal and flood stages, in kip-feet (see lateral loads). Use the larger value of total lateral forces for a normal or flood stage.*

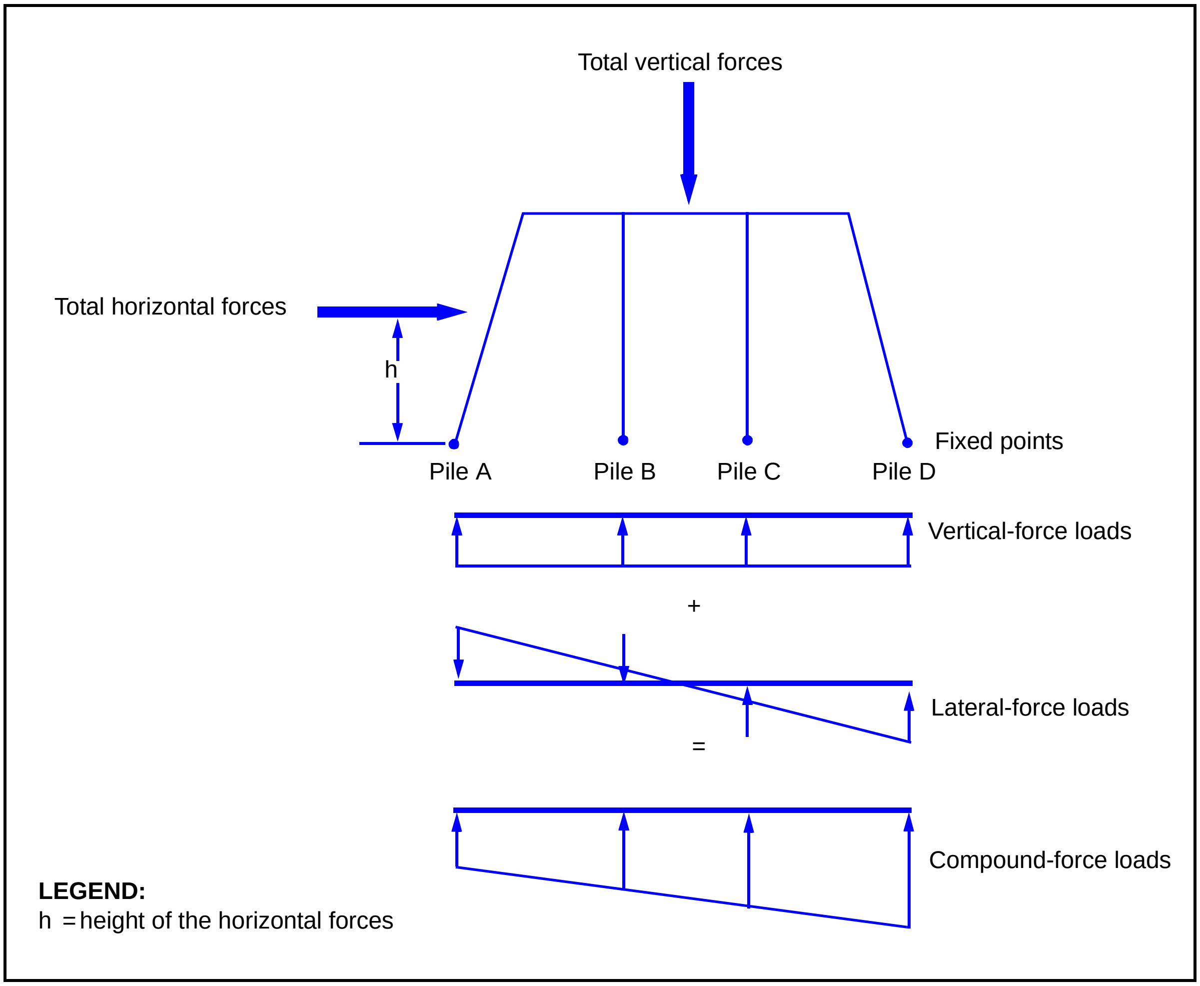


Figure 7-44. Combined Loading

S_p = *pile spacing, in feet (paragraph 7-111)*
N_{pr} = *required number of piles in the row or bent (equation 7-93)*

7-115. The allowable load on any friction pile in the group is computed as follows:

$$P_a = \left[\frac{N_{pe}}{N_{pr}}\right]P_B \qquad (7\text{-}95)$$

where—

P_a = *allowable load on a friction pile, in kips*
N_{pe} = *effective number of piles (Figure 7-43)*
N_{pr} = *number of piles required to carry the vertical load (equation 7-93)*
P_B = *allowable bearing capacity per pile, in kips (equation 7-15). For end-bearing piles, use the smaller of the bearing and buckling loads. For friction piles, use the smallest of the bearing load, buckling load, soil capacity, and driving capacity.*

7-116. For end-bearing piles, the allowable load on the outside pile is equal to the load per pile. For safety reasons, the actual load on the outside pile should be less than the load on a vertical pile. Compute the maximum actual load on an outside pile as follows:

$$P_m = \frac{P_{DL}}{N_p} + \frac{6\Sigma M}{S_p N_p (N_{pr} + 1)} \qquad (7\text{-}96)$$

where—

P_m = *maximum actual load on the outside pile, in kips*
P_{DL} = *total dead load, in kips (equation 7-56)*
N_p = *total number of piles in the group*
ΣM = *total moment of all lateral forces at the FP for both normal and flood stages, in kip-feet (see lateral loads). Use the larger value of total lateral forces for a normal or flood stage.*
S_p = *pile spacing, in feet (paragraph 7-111)*
N_{pr} = *total number of piles in the row or bent (equation 7-93)*

7-117. If the actual load on a single pile is negative, compare it with the allowable buoyancy force. Estimate this force as 40 percent of the soil bearing capacity, where the soil capacity is the allowable soil load *(equation 7-86)*. For safety, the actual load is less than or equal to the allowable buoyancy force. If the checks indicate that overload or buoyancy forces make the support unsafe, increase the pile spacing or add more piles or rows of piles.

PILE-PIER DESIGN

7-118. Pile-pier design is similar to pile-bent design except that two or more rows of piles are used. Also, pile piers require the design of a common-cap and corbel system as discussed below.

PILE-BENT DESIGN

7-119. A pile bent consists of a single row of piles with a pile cap. Brace bents to one another or to the adjacent abutment to reduce the unbraced length and to provide stability. Use the procedures described in *paragraphs 7-100* through *7-117* to design a pile bent, and use the procedures discussed below to design a pile cap. The consolidated process is as follows:

Step 1. Determine the loads acting on the bent.

Step 2. Determine the design capacity of a single pile (maximum allowable load).

Step 3. Determine the number of piles required.

Step 4. Determine the actual number of piles to be used based on group action.

Step 5. Check the combined loading produced by lateral loads and adjust the number of piles and spacing, if necessary.

Step 6. Design the cap by using the procedures outlined below.

CAP, CORBEL, AND COMMON-CAP DESIGN FOR PILE PIERS

7-120. To transfer a load from the superstructure to the supports properly, place a cap on each pile or post bent. If there are two or more bents in a single pier, use a common-cap and corbel system. A corbel is simply a short stringer connecting the bents *(Figures 7-45* and *7-46)*.

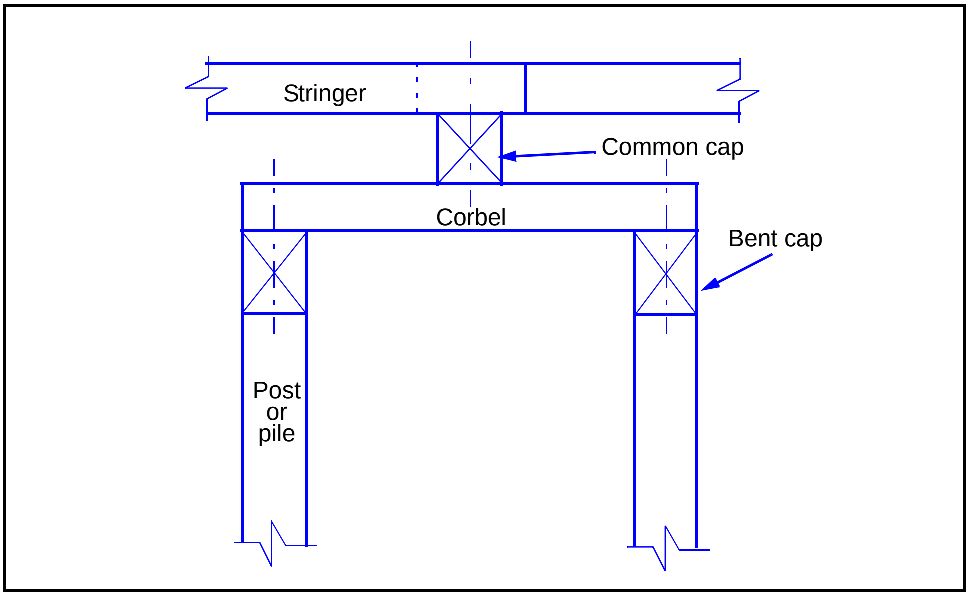


Figure 7-45. Two-Bent Pier With Common Cap

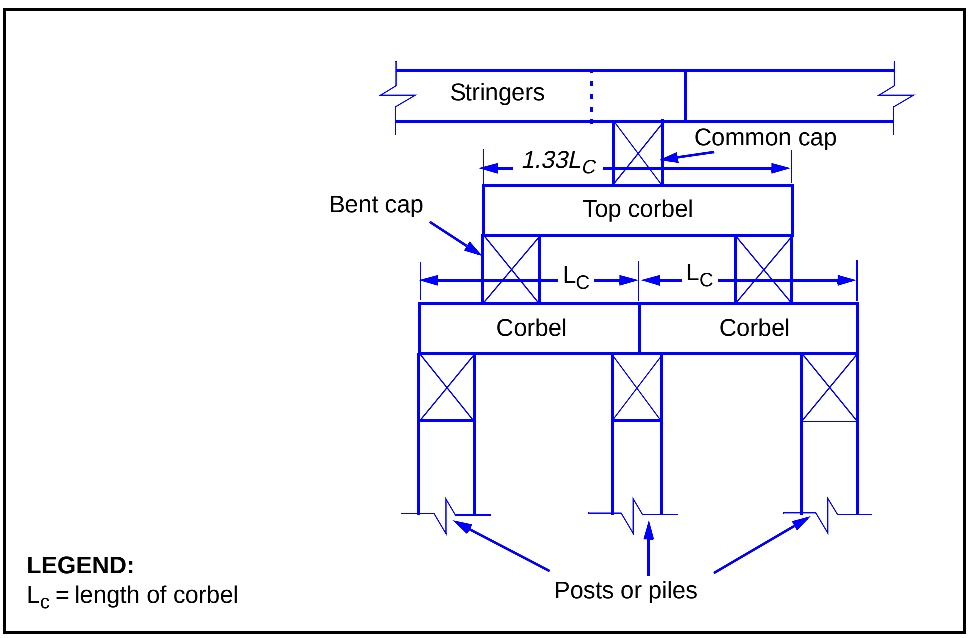


Figure 7-46. Three-Bent Pier With Common Cap

Cap Design

7-121. The minimum size for bent caps is 6 x 8 inches. The cap must also be at least as wide as the piles or posts supporting it. Check the bearing stress between the cap and the posts to ensure that the bearing stress does not exceed the maximum allowable stress for timber *(Table C-1, pages C-3 through C-6)*. Compute as follows:

- **Cap design load.**

$$P = P_{DL} + \frac{P_{LL}N_p}{4} \qquad (7\text{-}97)$$

where—

P = *total design load of the cap, in kips*
P_{DL} = *estimated substructure dead load of the pier, in kips (equation 7-56)*
P_{LL} = *estimated substructure live load of the pier, in kips (equation 7-57)*
N_p = *number of piles in the bent*

- **Actual bearing stress.** The actual bearing stress must not exceed the allowable bearing stress for the weakest member in the system. If the allowable bearing stress is exceeded, correct the system by adding more piles or posts or by adding timbers on the sides of the supports *(Figure 7-47)*.

$$F_B = \frac{P}{N_pA} \qquad (7\text{-}98)$$

where—

F_B = *actual bearing stress of the cap, in ksi*
P = *total design load of the cap, in kips (equation 7-97)*
N_p = *number of piles in the bent*
A = *cross-sectional area of the pile, in square inches (Table C-5, page C-10)*

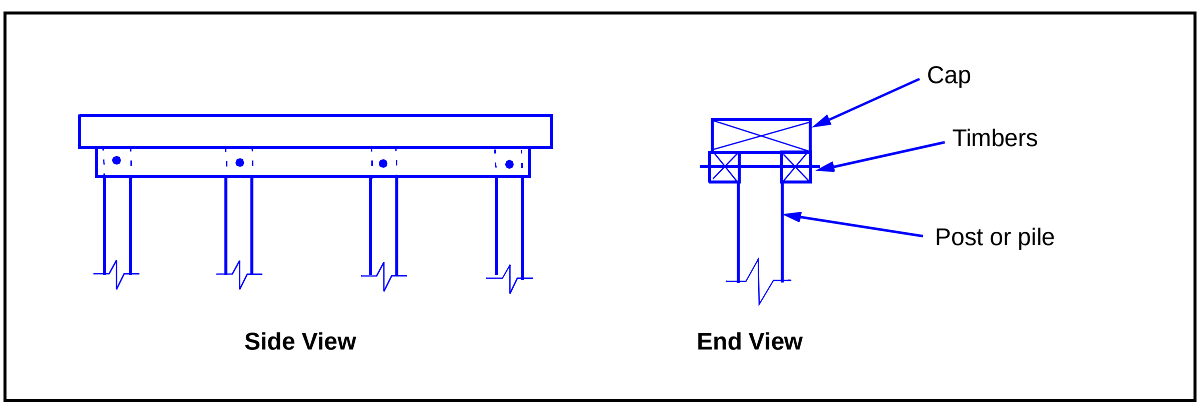


Figure 7-47. Timbers Added to Reduce Bearing Stress

- **Cap depth.** The depth of the cap must equal or exceed one-fifth of the spacing between the piles.

$$d_c \geq \frac{S_p}{5} \qquad (7\text{-}99)$$

where—

d_c = *cap depth, in feet*
S_p = *pile spacing, in feet (paragraph 7-111)*

Corbel Design

7-122. The corbel design process is different depending on the pier. The corbel design for two- and three-bent piers is discussed below.

7-123. **Two-Bent Pier.** The design length of each corbel equals the center-to-center spacing between bents. The actual corbel length exceeds the design length by a minimum of one cap width so that the corbel will have contact with the full area of each bent cap. Compute the required effective shear area and the required number of corbels as follows:

- **Required effective shear area.**

$$A_{vr} = \frac{P}{2F_v} \qquad (7\text{-}100)$$

where—

A_{vr} = *required effective shear area of the corbel system, in square inches*
P = *total design load of the corbel, in kips (equation 7-97)*
F_v = *allowable shear stress of the corbel, in ksi (Table C-1, page C-9)*

- **Required number of corbels.**

$$N_{cb} = \frac{A_{vr}}{A_v} \qquad (7\text{-}101)$$

where—

N_{cb} = *required number of corbels*
A_{vr} = *required effective shear area of the cap or corbel system, in square inches (equation 7-100)*
A_v = *effective shear area of the corbel, in square inches (Table C-4, page C-9)*

7-124. Shear normally governs corbel design. If the corbel length divided by the corbel depth is less than or equal to 12, do not consider shear controls and moment. Compute as follows:

- **Total shear.**

$$V = \frac{P}{2} \qquad (7\text{-}102)$$

where—

V = *total shear of the corbels, in kips*
P = *total design load, in kips (equation 7-97)*

- **Shear capacity.**

$$v = F_v A_v \tag{7-103}$$

where—

v = *shear capacity per corbel, in kips*
F_v = *allowable shear stress of the corbel, in ksi (Table C-1, pages C-3 through C-6)*
A_v = *effective shear area of the corbel, in square inches (Table C-4, page C-9)*

- **Number of corbels (shear).**

$$N_{cb} = \frac{V}{v} \tag{7-104}$$

where—

N_{cb} = *number of corbels*
V = *total shear of the corbels, in kips (equation 7-102)*
v = *shear capacity per corbel, in kips (equation 7-103)*

- **Spacing.**

$$S_{cb} = \frac{b_R}{N_{cb} - 1} \tag{7-105}$$

where—

S_{cb} = *center-to-center spacing of the corbel, in feet*
b_R = *curb-to-curb roadway width, in feet*
N_{cb} = *number of corbels (equation 7-104)*

- **Moment.** If moment governs the corbel design, compute as follows:

$$m = \frac{PL_{cb}}{4} \tag{7-106}$$

where—

m = *moment, in kip-feet*
P = *corbel design load, in kips (equation 7-97)*
L_{cb} = *corbel length, in feet (paragraph 7-123)*

- **Number of corbels (moment).** First, compute for the required section modulus:

$$S_{req} = \frac{m12}{F_b} \tag{7-107}$$

where–

S_{req} = *required section modulus*
m = *moment, in kip-feet (equation 7-106)*
F_b = *allowable bending stress of the corbel, in ksi (Table C-1)*

then, compute for the number of corbels:

$$N_{cb} = \frac{S_{req}}{S} \tag{7-108}$$

where—

N_{cb} = *number of corbels per bent*
S_{req} = *required section modulus (equation 7-107)*
S = *section modulus of one corbel (Table C-4)*

- **Spacing.**

$$S_{cb} = \frac{b_r 12}{N_{cb} - 1} \tag{7-109}$$

where—

S_{cb} = *corbel spacing, in inches*
b_r = *roadway width, in feet*
N_{cb} = *number of corbels (equation 7-108)*

7-125. **Three-Bent Pier.** A three-bent pier requires two corbel systems *(Figure 7-46, page 7-61)*, each designed separately. The design length equals the center-to-center bent spacing. The length of the top corbel equals one and one-third times the length of the bottom corbel. The bottom corbels are not continuous; they are simply supported at the center bent. See *equations 7-104* and *7-105* for top corbels. If the ratio of 4 times the design length to 3 times the pile depth is less than or equal to 12, shear governs. If moment governs, see *equations 7-107* and *7-108*.

Common-Cap Design

7-126. The minimum size for a common cap is 6 x 8 inches. Compute the common-cap width and depth as follows:

- **Common-cap width.**
  - **Top common cap.** Compute the minimum width of the common cap for a two-bent pier or the top common cap of a three-bent pier as follows:

$$b_c = \frac{p}{2N_{cb}b_{cb}F_c} \tag{7-110}$$

where—

b_c = *cap or sill width, in inches*
P = *total load, in kips (equation 7-97)*

N_{cb} = *number of corbels in the top bent (equation 7-108)*
b_{cb} = *corbel width of the top cover, in inches*
F_c = *allowable bearing of the supports perpendicular to the grain of the support material (Table C-1, pages C-3 through C-6)*

- **Bottom common cap.** Compute the minimum width of the bottom common cap of a three-bent pier as follows:

$$b_c = \frac{P}{N_{cb} b_{cb} F_c} \quad (7\text{-}111)$$

where—

b_c = *minimum width of common cap, in inches*
P = *corbel design load, in kips (equation 7-97)*
N_{cb} = *number of corbels in the lower level (equation 7-108)*
b_{cb} = *corbel width in the lower level, in inches*
F_c = *allowable bearing of the cap or corbel, perpendicular to the grain of the cap or corbel material, in kips (Table C-1)*

- **Common-cap depth.** The depth of any common cap must equal or exceed one-fifth the spacing of the corbels supporting it. Compute as follows:

$$d_c > \frac{S_{cb}}{5} \quad (7\text{-}112)$$

where—

d_c = *cap depth, in inches*
S_{cb} = *corbel spacing, in inches (equation 7-109)*

TIMBER- AND STEEL-FRAMED TOWERS

7-127. The AFCS contains design drawings for timber- and steel-framed towers.

CONCRETE PIERS

7-128. Concrete piers may be constructed of either mass or reinforced concrete. Mass-concrete piers can be built in about the same amount of time as reinforced concrete piers but are not economical because they require such large quantities of concrete. Since concrete piers are normally used in permanent bridges, this FM does not go into detail on their design.

SHALLOW FOUNDATIONS

7-129. When shallow bedrock prevents pile installation, use some other type of specially designed shallow foundation. The paragraphs below describe a few shallow piers (other than standard timber and steel piers) that have been successful in the field.

Rock Footings

7-130. In dry gaps or shallow streambeds, construct rock footings as shown in *Figure 7-48*. Hold the rocks in place by driving steel pickets into the soil. Dump the rocks into the steel enclosure and level them at the proper elevation. To provide a smooth surface for timber or steel piers, place a layer of concrete over the rocks. Settlement may be a problem since the rocks have to be placed directly on mud. In this case, put a surcharge of heavy material on the rock footing to induce settling before leveling and capping. After the footing settles, remove the surcharge and level and cap the rock footing. The total load acting on the footing (including the weight of the footing itself) divided by the planned area of the footing must not exceed the bearing capacity of the local soil.

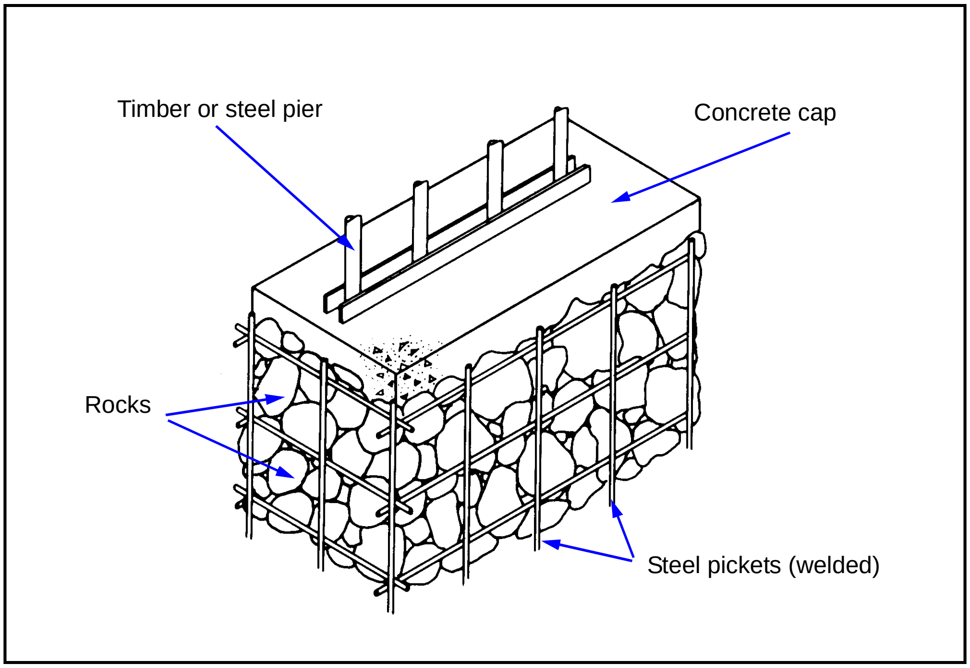


Figure 7-48. Rock Footing

Concrete Footings

7-131. In shallow watercourses with firm beds, use a concrete footing. Place the forms and anchor them with weights so that they will not float away. Then use a tremie to place concrete in the forms. Place the footing only when the temperature of the water will not fall below 45°F during the curing process.

Steel or Timber Footings on Rock or Concrete

7-132. Embed steel or timber posts directly in the concrete or on top of the rock or concrete footings.

Steel-Culvert Pipe

7-133. Steel-culvert pipes can serve as expedient bridge piers. Fill the pipes with rocks, cap them, and place them on edge. At least two rows of pipe are needed, with both lateral and longitudinal bracing to provide stability. This type of pier requires a cap and corbel system. Weld the steel pipe together as rigidly as possible to provide one continuous pipe. Piers of this type should not exceed 12 feet in height and can be placed on a rock or concrete footing. Do not use concrete pipes since sections are short and tend to be unstable when stacked.

FOOTINGS AND GRILLS

7-134. The AFCS contains descriptions on the use of footings and grills as special foundations for steel or timber towers. See *FM 5-277* for more information on grills.

CULVERTS

7-135. Reinforced concrete and corrugated-metal culverts are often used in shallow or intermittent watercourses in place of bridges. See *FM 5-430-00-1* for information on culvert designs in connection with drainage.